LANDSLIDE INVESTIGATION AND HAZARD ZONATION IN THE GREYMOUTH URBAN AREA.

A thesis
submitted in partial fulfilment
of the requirements for the degree
of
Master of Science in Engineering Geology
at the
University of Canterbury
by

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1993

ABSTRACT.

Greymouth township is located at the outlet of the Grey River on the West Coast of the South Island. Urban expansion is currently encroaching on the steep (20+ °) hill country underlain by westerly dipping Nile Group limestones and massive Blue Bottom Group mudstones. This study was an engineering geological investigation and landslide hazard assessment of the Greymouth urban area. The purpose of such a study was to identify areas prone to landsliding in order to establish areas that are suitable for future urban development.

Landslides were identified from aerial photographic interpretation and engineering geological mapping at a scale of 1:10 000 and were classified as translational rock slides and rock block slides, falls, rock avalanches, translational debris slides, rotational debris slumps and flows. Limited geotechnical testing was undertaken for selected rock and soil lithologies with the objective of characterising bedrock and soils in the area.

Two landslides, a translational rock slide (Stanton Crescent Slide) and a translational debris slide (Australasian Hotel Slide), identified by the author were mapped in detail at scales of 1:1500 and 1:1000 respectively. The mode of failure for both of these landslides was determined and general failure models were developed from field investigation.

From an historical data base compiled as part of this study, it was concluded that intense or prolonged rainfall is the primary initiating agent in the development of landslides involving surficial materials. Field mapping suggests that bedding within the mudstone is an important defect along which failure takes place. Earthquakes have also been important in initiating landslides and will be in the future.

An engineering geological data base including engineering geological mapping (1:10 000), aerial photographic interpretation, geotechnical characterisation and the information that was obtained by the two site specific studies was compiled, from which landslide hazard zonation and land-use suitability maps at a scale of 1:10 000 were derived. These identify the most landslide prone areas as the coastal escarpment and other slopes underlain by mudstone and the Twelve Apostles Range. In addition, these maps provide a guide as to the geotechnical limitations to development and the level of additional site investigation that may be required in a given area.

Recent changes in government legislation has seen a growing awareness by local administering authorities of the need for sound engineering geological information on which to base land-use planning. It is anticipated that the local authority will use the information contained in this study for the administration of land-use planning in the Greymouth urban area.

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My foremost gratitude must go to my parents Lena and Lawrie for their financial support, help and encouragement over the last five and half years. Words cannot express my appreciation of the effort that you have put into my education.

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I would like to thank to my supervisor, David H. Bell for his helpful comments during the thesis compilation, review of drafts and for the general support during the term of the project. Thanks also to Jarg Pettinga for editing drafts at short notice. I am also grateful to the staff of the Geology Department for their assistance and co-operation during the term of the project.

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I salute those of you who grovelled around in the mud while undertaking field work in the name of science and often at very short notice, namely: Nick McIvor, Grant Meynell, Craig Philps and Wayne Stephenson. For without you guys, the field work would have been a far greater and begrudging task. I feel that it must be time to cash in all those promissory notes, redeemable at the "Bush" only!!

To the "class of 1993", I extend my warmest appreciation of the good times and laughs over the last four and a half years. The names are too many to mention but none of you will be forgotten. I wish all of you the very best in the years to come.

To Aileen Lawrie, I extend my sincerest gratitude for all that you have done for me over the last 18 months. Your total support, help, encouragement and editing of my thesis drafts has been the major contributing factor in my ability to complete "this thesis".

CHAPTER 1.
INTRODUCTION.

1.1 BACKGROUND TO THE STUDY.

Greymouth is the main town and administrative centre on the West Coast of the South Island (Figure 1.1). Historically, the township was sited on the flood plain near the mouth of the Grey River and serviced a major part of the region. Throughout much of Greymouth's history, major floods have repeatedly affected the commercial centre of the town and the surrounding older residential areas located on the flood plain adjacent to the river. Despite the completion of a flood protection scheme in 1988, there has been a growing trend to develop the "landslide-prone" hill country to the east and south of the town, thereby avoiding potential inundation.

The West Coast Regional Council is currently assessing natural hazards in terms of the future planning requirements for the township, under the impetus of the recently introduced Resource Management Act 1991 (RMA). This study is an assessment of the "landslide hazard" in the Greymouth urban area and forms an integral part of the overall hazard assessment that has been undertaken by the council. In the course of this study, detailed investigation was carried out into specific landslides in the Greymouth area that were identified by the author (Chapter 4). Information generated by these site specific studies was then extrapolated to similar landslides in comparable physical settings. The hazard posed by each landslide could then be evaluated, and zonation maps produced (Chapter 5).

The West Coast Regional Council (TWCRC) has certain legal responsibilities with regard to natural hazards (including landslides) under Sections 30(1)(c)(iv) and 35(5)(j) of the RMA as discussed further in Chapter 5. This study aims to assist TWCRC in meeting its statutory obligations under the RMA through:

1) the identification of existing landslide features;

2) the delineation of areas that may be susceptible to instability in the future; and

3) the development of a landslide hazard zonation scheme for the mapped area that is suitable for the administration of urban planning.

The information has been diagrammatically presented in Chapter 3 as a landslide inventory from which landslide hazard zonation and land use capability maps have been derived and which are presented in Chapter 5. The mapping scale is 1:10 000, which allows both broad scale engineering geological relationships and relatively detailed information to be displayed.

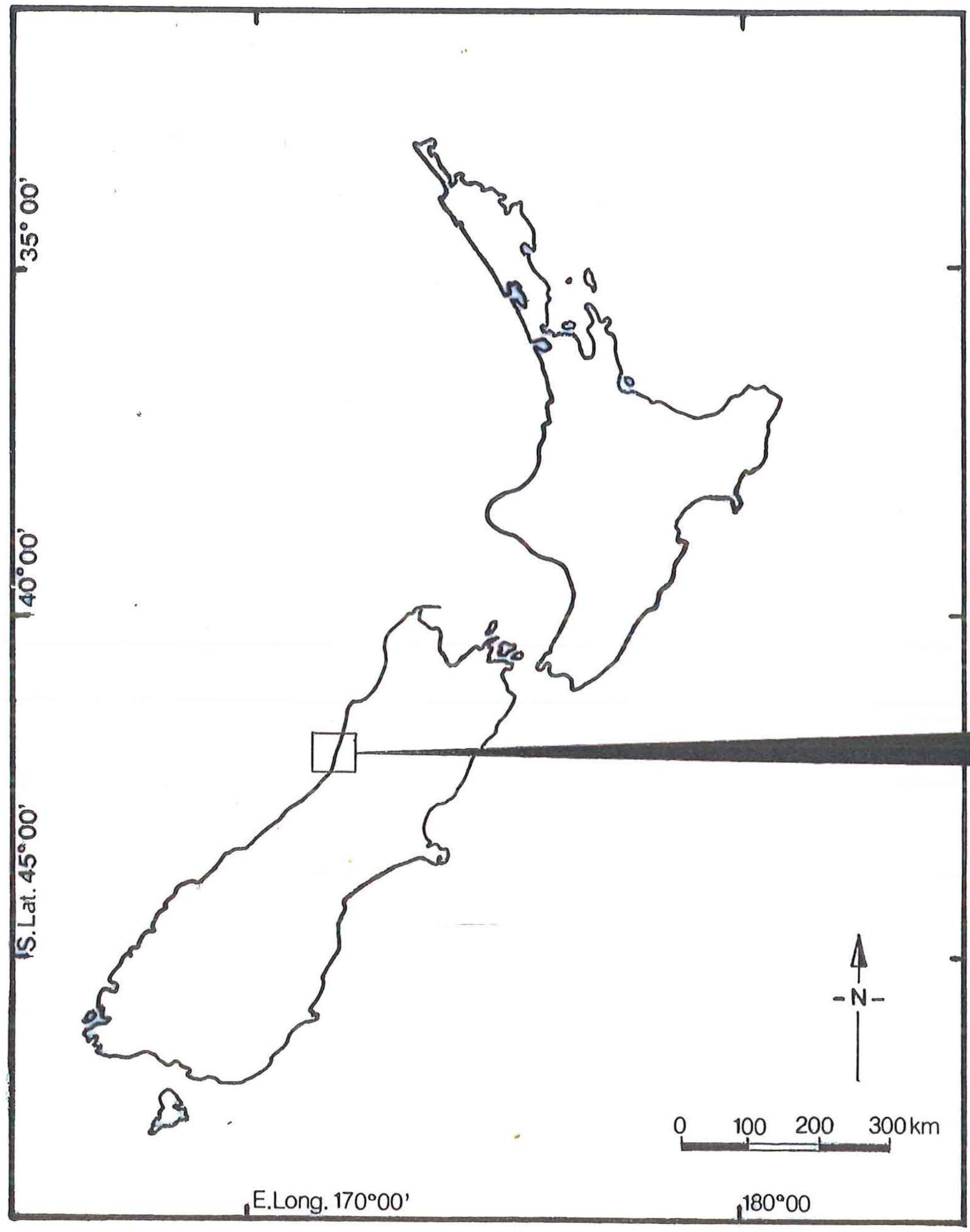


FIGURE 1.1 LOCATION OF THE FIELD AREA.
SOURCE: DEPARTMENT OF LANDS AND SURVEY, 1:250,000 SERIES TOPOGRAPHICAL MAP. NZMS 262 SHEET 10. GREY.

1.2 SPECIFIC THESIS OBJECTIVES.

The principal objectives of the thesis are:

1) to carry out an engineering geological investigation of the study area, to determine geologic relationships and identify existing areas affected by landslides;

2) to determine geotechnical properties of bedrock and soil materials relevant to a landslide assessment;

3) to develop engineering geological models for identified failure types;

4) from existing data sources, to compile an historical account of landslides that have affected the Greymouth area; and

5) to develop a landslide hazard zonation for the field area at a scale of 1:10 000.

With these five objections in mind, a work programme involving background research commenced in February 1992, with field work starting in late May 1992 and continuing through until May 1993. Essentially a field mapping-based project, subsurface investigation was carried out in localised areas to supplement the surface information generated.

1.3 STUDY AREA DESCRIPTION.

1.3.1 PHYSIOGRAPHY.

Greymouth township is situated on a narrow coastal plain extending from immediately south of Point Elizabeth to beyond the southern limit of the field area (Figure 1.1). Steep hill country including the Twelve Apostles Range and Peter Ridge confines the coastal plain in the east (Figure 1.2). Tectonically uplifted marine terraces at 6-9m above sea level (forming a higher section of the coastal plain), and at 70-80m above sea level (*a.s.l.*) are present north and south of the Grey River.

The Twelve Apostles Range and Peter Ridge have formed on erosion-resistant Tertiary limestone. Drainage in these two ranges has incised along an east/west trending joint set resulting in steep sided, V shaped valleys. The Twelve Apostles Range reaches a maximum altitude of about 330m *a.s.l.*, whilst Peter Ridge only attains a height of around 270m *a.s.l.* within the field area.

Part of the Greymouth township extends south-east towards Boddytown (Figure 1.1) along a broad river valley that has been eroded by Sawyers Creek into marine mudstone. Topography formed within the mudstone is more subdued than the karstic topography of the Twelve Apostles Range and Peter Ridge. Maximum relief in the south and south-east of the field area rarely exceeds 100m *a.s.l.*.

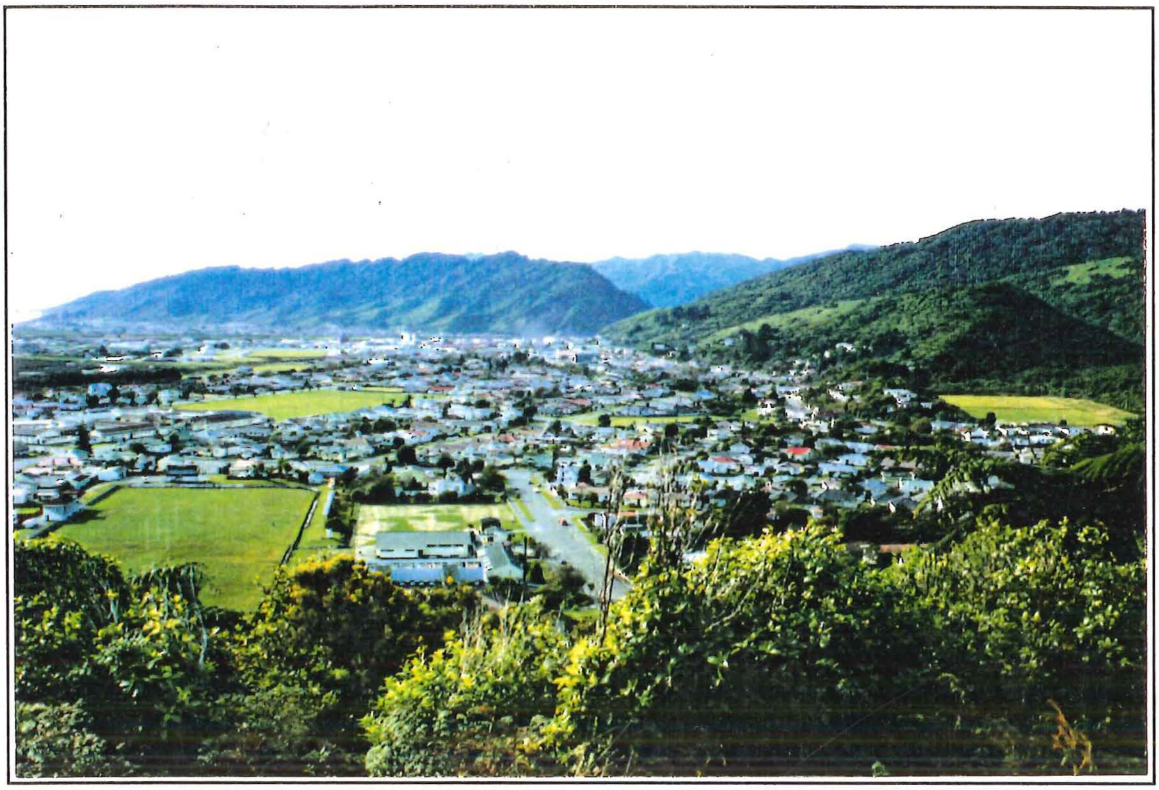


Figure 1.2 Contrasting physiography, looking north from Arnotts Heights towards the Tweleve Apostles Range and the northern extent of Peter Ridge. The Grey River flows out through the gap in the ranges, centre right.

1.3.2 CLIMATE.

Westland's climate is characterised by a high annual rainfall caused by the orographic forcing of the prevailing westerly airstream over the Southern Alps. Rainfall (at the Greymouth aerodrome *c.* 4m *a.s.l.*) is around 2500mm per annum (N.Z. Met. Service 1987), much of which occurs during high intensity rainstorms (Table 1.1). These commonly have a dramatic destabilising effect on the landscape. The prevailing westerly winds are warm and when combined with the 1700 sunshine hours received per annum, produce a mild climate with mean annual temperatures of around 12 °C.

Table 1.1 Rainfall Depth - Duration - Frequency Relations for Greymouth

Duration	10M	20M	30M	1H	2H	6H	12H	24H	48H	72H
T = 2	12	17	21	30	41	64	84	109	136	153
T = 5	16	23	30	43	59	91	118	146	185	205
T = 10	19	27	36	51	71	109	141	170	217	240
T = 20	21	31	42	59	83	127	162	193	248	273
T = 50	25	36	50	70	98	149	190	224	288	316

The figures are in millimetres, T = 2-50; computed values of rainfall depth for return periods 2 to 50 years, M = minutes and H = hours. Table modified from data presented in Coulter and Hessell (1980).

1.3.3 VEGETATION.

The Greymouth area was originally covered by thick native forest which extended in a continuous belt from the Marlborough Sounds to Stewart Island (Cockayne 1967). Mining activities around the town in the late 1800's stripped much of the original forest cover and more recently, ridge top haulers largely completed the devegetation process.

At present most of the coastal plain, inland river flats and elevated terrace surfaces are vegetated by a combination of native and introduced grasses and scrub varieties. The steep hill country is covered by regenerating native forest with virgin bush present at the range tops. Introduced species dominate the low-lying areas, but native varieties predominate in elevated areas. A list of common plant species (both native and introduced) found in the area (L.J. Metcalf 1993 *pers. comm.*) is given in Appendix A1.1.

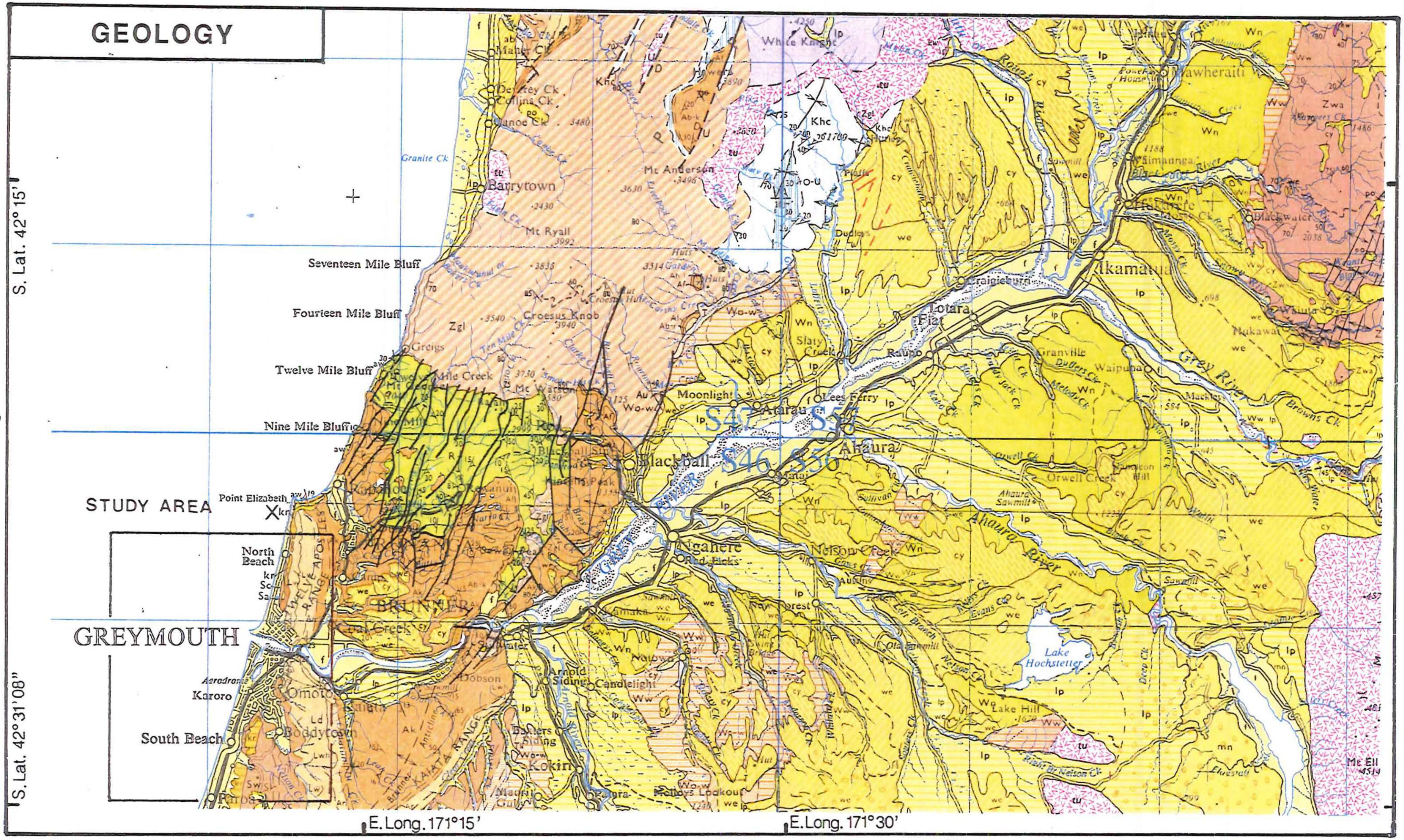
1.3.4 PRESENT LAND-USE.

Urban development (hereby taken to include both industrial, residential and associated engineering works) is concentrated immediately around the Grey River and extends discontinuously north and south along the narrow coastal plain, and southeast towards Boddytown (Figure 1.1). Locally, subdivisions have expanded up a coastal scarp (Stanton Crescent, Freyberg Terrace, Tindale Street) and encroached on the terrace surfaces above (Arnotts Heights).

Extensive pastoral farming occurs in low-lying areas of flat to gently inclined land and intensive agriculture (for example dairy farming in the Range Creek area) exists where suitable soil conditions are found. Exotic forestry operations are present inland from Paroa and South Beach. Elsewhere, much of the forested areas are zoned reserve land (Rapahoe Range Scenic Reserve, Aorangi Scenic Reserve and Marsden Point Scenic Reserve).

1.4 REGIONAL GEOLOGICAL SETTING.

The Greymouth field area is located within the structurally complex and tectonically active West Coast geologic region which extends from Fiordland to North West Nelson (Nathan *et al.* 1986). Paleozoic basement strata within the region includes the tectonically deformed Greenland Group (Figure 1.3), a distinctive green coloured alternating mudstone and sandstone sequence, and intrusive granites of Paleocene to mid Cretaceous age (Tulloch 1983). Faulted and folded mid Cretaceous and Cenozoic cover rocks (Figure 1.3) are generally considered to represent a post rifting sedimentary sequence deposited following the separation of Australia and New Zealand during the mid Cretaceous (Laird 1993). The regional stratigraphy and history of the West Coast is further reviewed in Appendix A1.2, and a geologic time scale is provided in Appendix A1.3.


GEOLOGY
STUDY AREA
GREYMOUTH
S. Lat. 42° 15'
S. Lat. 42°31'08"
E. Long. 171°15'
E. Long. 171°30'
Seventeen Mile Bluff
Fourteen Mile Bluff
Twelve Mile Bluff
Nine Mile Bluff
Point Elizabeth
North Beach
Aerodrome
Karoro
South Beach
Barrytown
Mt Ryall
Croesus Knob
Mc Anderson
White Knight
Granite Ck
Greigs
Moonlight
Atarau
Lees Ferry
Blackball
Ahaura
Ngahere
Red Jacks
Nelson Creek
Totara Flat
Raupo
Craigieburn
Ikamatua
Granville
Waipuna
Grey River
Browns Ck
Orwell Creek
Ahaura River
Lake Hochstetter
Lake Hill
Arnold Siding
Candlelight
Bakers Siding
Kokiri
Molloys Lookout
Dobson
Omoto
Boddytown
Kaiata Range
Brunner
Coal Creek
Blackwater
Waiuta
Hukawai
Nawhiraiti
Mc Ell
Slaty Creek
Dudleys
New Forest
S47
S57
S46
S56

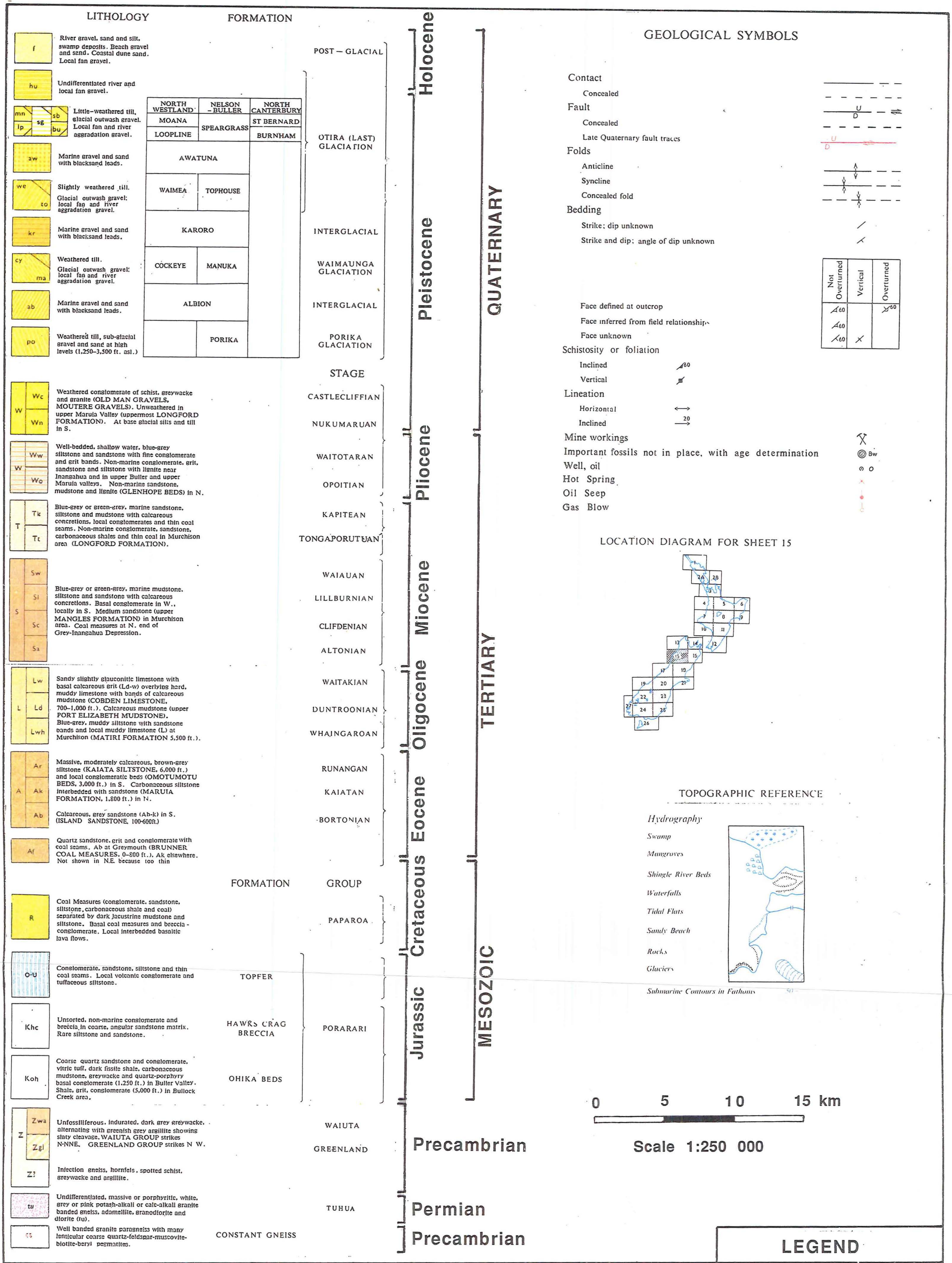


FIGURE 1.3 WEST COAST GEOLOGY. From Bowen (1975) NZGS Sheet 15 Buller

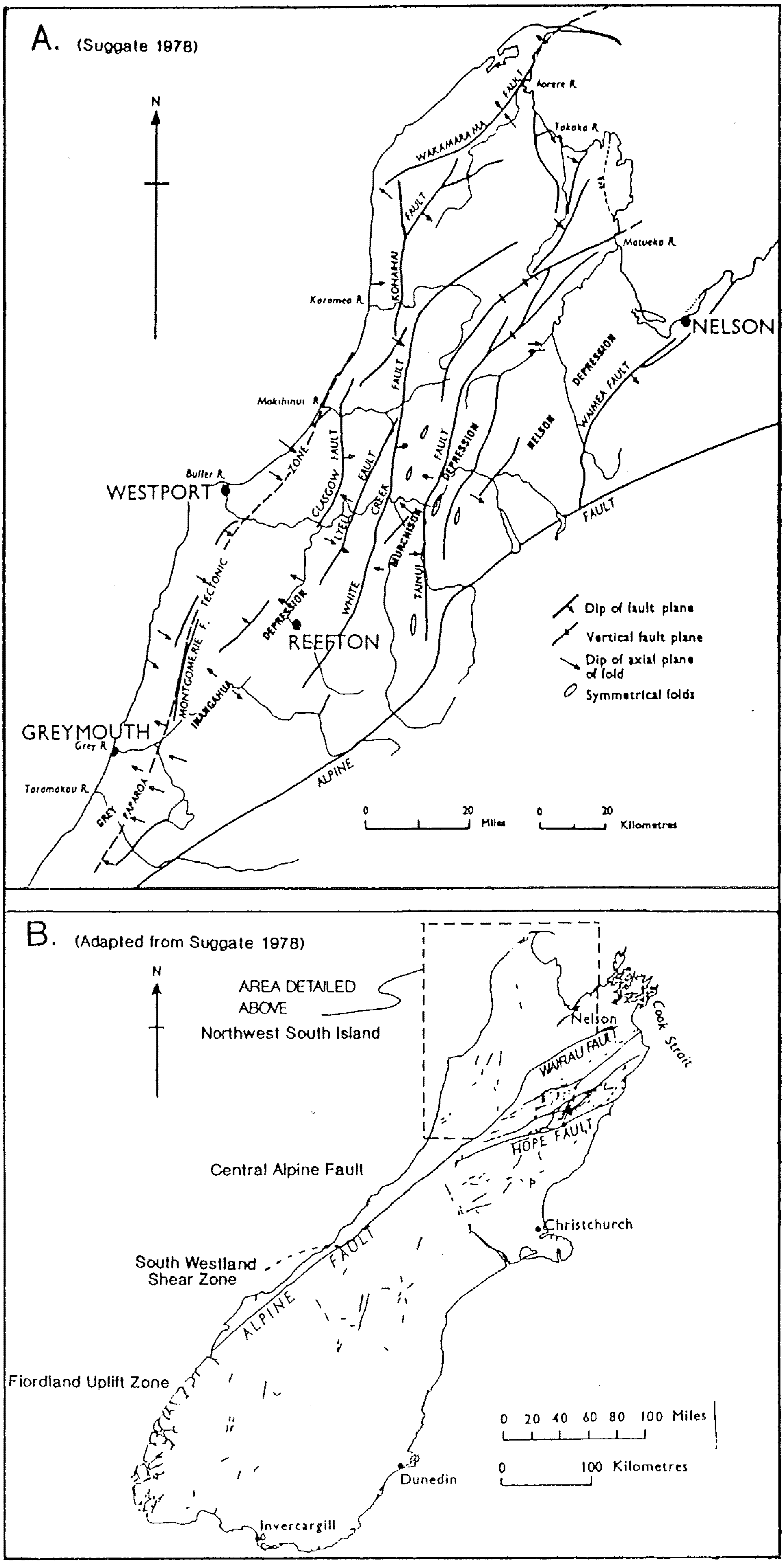


Figure 1.4 Main structural features in the West Coast Region. After Suggate (1978).

The West Coast region is situated to the west of the Alpine Fault, a broad shear zone (200 - 1500m wide) representing a transform plate boundary between the Indian - Australia plate to the west, and the Pacific plate to the east. Movement along the fault zone extending from Milford Sound, (Fiordland) in the south west, to Cloudy Bay, (Marlborough) in the north east is predominantly dextral strike slip with an oblique compressional component. The compressional component of the tectonic strain is responsible for the N-NNE alignment of the main structural features within the region (Figure 1.4) and is being accommodated by reverse slip on pre-existing faults (Beanland and Anderson 1992).

1.5 TERMINOLOGY.

Relevant definitions for various terminology used in the report are provided below. It is not the purpose of these sections to debate the merits of each definition, but rather the clarification of their meaning. The terminology, classification of landslides and the assessment of slope instability has been reviewed further in Appendix A1.4.

1.5.1 ENGINEERING GEOLOGY, ROCK AND SOIL MASS AND MATERIAL.

"Engineering geology" may be defined as both the evaluation of active processes potentially affecting a given site, and the systematic recording of foundation conditions (Bell 1979). This forms the basis of the approach that has been used in this landslide investigation and hazard zonation of the Greymouth area.

In engineering geological practice there is a fundamental distinction drawn between the rock or soil "material" and the rock or soil "mass". The term "material" introduced by Duncan (1969) refers to "*intact rock or soil that is composed of mineral grains or crystals and the associated void or pore space, which may be air or water-filled*". The term "mass" is applied to "*rock or soil that is not effectively homogeneous. Inhomogeneity may result from the presence of two or more materials (rock lithologies or soil "substances") and/or the existence of geological discontinuities which disrupt the rock or soil material*" (Bell and Pettinga 1983).

The terms "rock" and "soil" have different definitions in geological and engineering geological usage, therefore some clarification of these terms is necessary. For the purpose of this study, the engineering geological terminology has been adopted. Rock material therefore, is defined as:

> *"hard and rigid, requiring blasting or some other equivalent method for excavation and is not significantly affected by immersion in water" (Bell and Pettinga 1983 p. 14.5).*

Soil material is defined as:

> *"Loose or soft deposits that can be excavated by normal earth moving equipment and will disaggregate or become remouldable on immersion in water" (Bell and Pettinga 1983 p.14.5).*

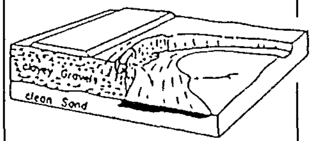
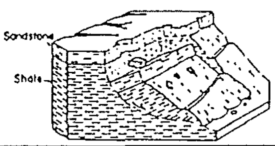
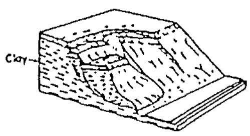
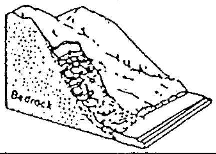
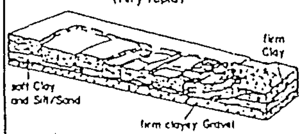
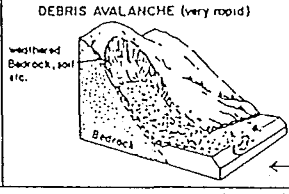
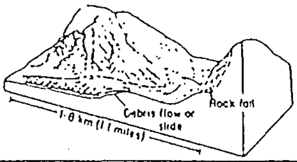
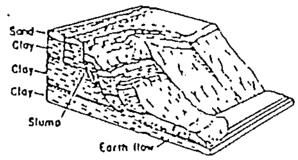
"Bedrock" is defined as the first rock unit of pre-Quaternary age present beneath the Earth's surface, and "surficial geology" is taken to include all unconsolidated sediments at or near the Earth's surface.

The classification scheme proposed by Bell and Pettinga (1983) has been used in this study for the specific purpose of recording field descriptions of the rock and soil mass and material properties of the various units within the study area. Copies of the sheets that were used in the field are provided in Appendix A1.5.

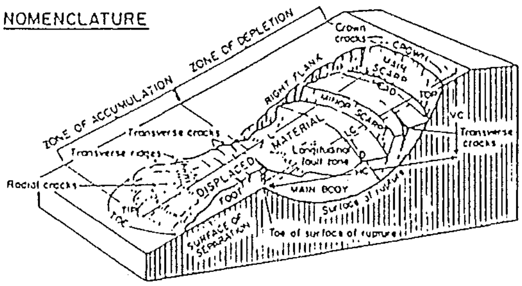
1.5.2 SLOPE STABILITY.

Landslides are included in the group of slope movements wherein shear failure occurs along a specific surface or combination of surfaces (Schuster 1978) and involve the predominantly downslope transfer of geological materials under the influence of gravity (Bell 1990). A number of authors have advanced and endorsed a variety of different terminology in the literature, hence there is a need to provide some initial definitions. To avoid confusion, it is considered appropriate to follow the terminology of Varnes (1978). Varnes (1978) recognised "slope movements" as the correct technical term whilst continuing to use "landslides" with the same meaning. "Slope movements" (and landslides) thus involve "*... the essential downslope transfer of geological materials under the influence of gravity*". The terminology developed by Varnes (1978) has been adopted for this study and the terms "slope movements" and "landslides" have been used synonymously through-out the text.

The classification developed by Varnes (1978) for the purpose of describing landslides is widely used and accepted internationally. Varnes (1978) recognises six fundamental types of movement and these may be grouped into slides; flows and lateral spreads; falls and topples; and complex slope movements (Figure 1.5). "Sliding" motion involves shear displacement along one or more discrete failure surfaces; "flows" involve an internal shearing motion resembling that of viscous fluids; "falls" involve a component of free fall in their motion, while "topples" are essentially a pivotal motion, about a centre of rotation. In many cases it is common to discover that failure may include a combination of these movements, and such slope failures are termed complex slope movements.

Type of Movement		Type of Material (before movement): BEDROCK	ENGINEERING SOILS: Predominantly Coarse	ENGINEERING SOILS: Predominantly Fine
I FALLS free fall through the air, with leaping, bounding or rolling of fragments		ROCK FALL (extremely rapid) 	DEBRIS FALL	EARTH FALL
II TOPPLES pivotal rotation about centre of gravity (⟶ fall or slide)		ROCK TOPPLE	DEBRIS TOPPLE 	EARTH TOPPLE
III SLIDES shear displacement along one or several discrete surfaces, or within a relatively narrow zone	A ROTATIONAL	ROCK SLUMP (extremely slow to moderate) 	DEBRIS SLUMP	EARTH SLUMP 
	B TRANSLATIONAL	ROCK BLOCK SLIDE ROCK SLIDE (both defect-controlled)	DEBRIS SLIDE (v. slow to rapid) 	EARTH BLOCK SLIDE EARTH SLIDE
IV LATERAL SPREADS lateral extensional movements A. without a basal shear surface B. due to liquefaction		ROCK SPREAD	DEBRIS SPREAD	EARTH SPREAD (very rapid) 
V FLOWS extremely slow and non-accelerating movement in bedrock; slow to rapid viscous (= fluid-like) movements in soils		ROCK FLOW (deep creep)	DEBRIS AVALANCHE (very rapid) 	EARTH FLOW ⟵ SOIL CREEP ⟶
VI COMPLEX involves combination of two or more principal types of movement		ROCK FALL - DEBRIS FLOW (extremely rapid) 	SLUMP - EARTH FLOW 	

NOMENCLATURE



RATE OF MOVEMENT SCALE

Rate	Class
	extremely rapid
3 0m/sec	
	very rapid
0 3m/min	
	rapid
1·5 m/day	
	moderate
1·5m/month	
	slow
1 5m/year	
	very slow
0 06m/year	
	extremely slow

SLOPE MOVEMENT TERMINOLOGY AND CLASSIFICATION
(after Varnes, 1978)

FIG. 1.5

Furthermore, the Varnes (1978) landslide classification is subdivided in terms of the materials involved in slope movement (Figure 1.5). Specifically, *in situ* "bedrock" and "engineering soil" are recognised, the latter being further subdivided into "debris" (coarse grained engineering soils) and "earth" (fine grained engineering soils). Debris by definition contains at least 20% of fragments finer than 2mm whilst earth contains 80% finer than 2mm. A graduated movement scale (Figure 1.5) ranging from extremely slow (≤0.06m/year) to extremely rapid (≥3.0m/second) also forms an integral part of the Varnes' classification scheme.

1.6 THESIS ORGANISATION.

Chapter 2 reviews the geology and geomorphological evolution of the field area and thereby sets a framework within which a landslide hazard assessment can be made.

Chapter 3 details the engineering geological investigations that were carried out and presents the results of the geotechnical characterisation of rock and soil lithologies with particular reference to slope stability.

Chapter 4 presents the results of two site specific studies and develops engineering geological models for these failure types.

Chapter 5 reviews the principles of landslide hazard zonation and some of the common hazard zonation schemes in use both here and overseas. The author then outlines the recommended landslide hazard zonation scheme that has been developed for the Greymouth area. The author's recommendations to the TWCRC and areas for future research are outlined in this chapter.

Chapter 6 presents the summary and conclusions of this thesis and reiterates the author's recommendations to TWCRC.

CHAPTER 2.
GREYMOUTH GEOLOGY.

2.1 INTRODUCTION.

The purpose of this chapter is to briefly review the geology of the field area as compiled from the available published literature insofar as this is relevant to a landslide hazard zonation. For this purpose, the geology has been dealt with under three main headings: "Stratigraphy"; "Structure and Tectonics"; and "Geomorphological Evolution". The importance of these for land use planning is then discussed under the heading "Slope Stability Implications".

The most up to date sources of published information on the geology of the West Coast are sheets S23 and S30 (Nathan 1975), S37 (Laird 1988), S44 (Nathan 1978), their accompanying notes and Nathan *et al.* (1986). For greater detail on the geology of the West Coast the reader is referred to these. The stratigraphic subdivisions of Nathan (1978) have been adopted in this landslide assessment, and much of the following review is also based on this source.

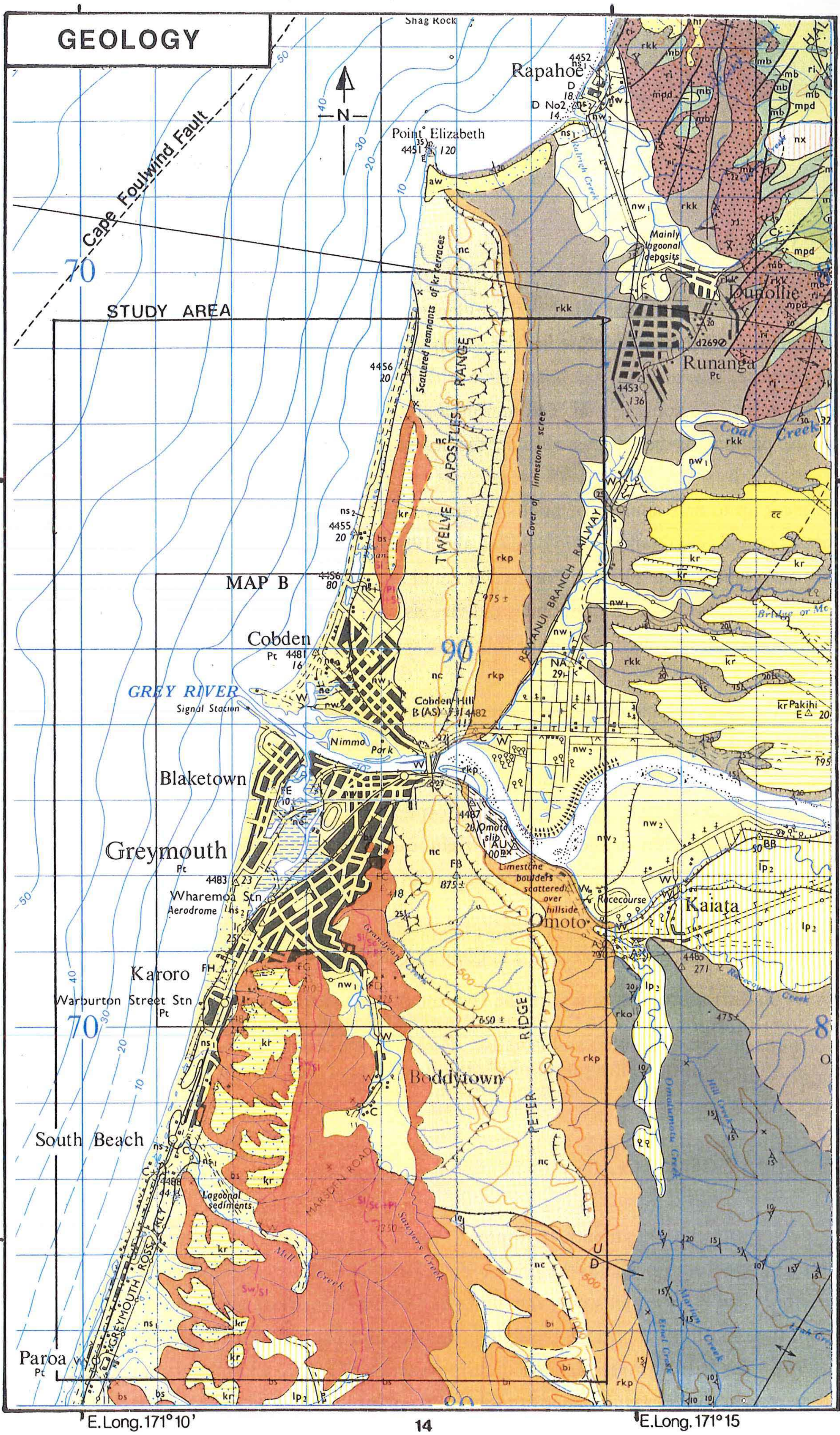
Despite the large volume of geological literature available, there is little published engineering geological information available and almost none specific to the Greymouth area. Although numerous small site specific investigations have been carried out, published sources are rare and with recent changes in many of the former government departments, much of the information is now unavailable due to copyright restrictions. Of the few studies that have been carried out around Greymouth two of the more relevant are:

1) An investigation by the DSIR of the Omoto Slip inland from Greymouth (Paterson 1989), a summary of which is presented in Chapter 4; and

2) A land-use capability study for the Greymouth area carried out by Hutchison and Mckie (1979) for the Westland Catchment Board. This study presented useful information on slope angles, vegetation and land use. However, the report, discussed further in Chapter 5, contains little engineering geological information of relevance to this study.

2.2 STRATIGRAPHY.

The geology of the Greymouth field area (Figure 2.1) comprises a sedimentary sequence ranging in age from Eocene to the present day. Bedrock consists of the Nile Group (Cobden Limestone) and marine mudstones that underlie (Kaiata Formation) and overlie (Blue Bottom Group) the limestone. Surficial geologic units unconformably overlie bedrock and comprise Quaternary sands and gravels deposited during a series of glacial and interglacial episodes, as well as regolith and colluvial deposits formed from the weathering of the underlying parent materials.



GEOLOGY
Cape Foulwind Fault
STUDY AREA
MAP B
Shag Rock
Rapahoe
Point Elizabeth
Dunollie
Runanga
Cobden
GREY RIVER
Signal Station
Nimmo Park
Blaketown
Greymouth
Wharemoa Stn
Aerodrome
Karoro
Warburton Street Stn
Boddytown
South Beach
Paroa
Cobden Hill
Omoto
Omoto slip
Kaiata
Racecourse
Pakihi
TWELVE APOSTLES RANGE
PETER RIDGE
RAPANUI BRANCH RAILWAY
GREYMOUTH ROSS RLY
MARSDEN ROAD
Mainly lagoonal deposits
Lagoonal sediments
Cover of limestone scree
Limestone boulders scattered over hillside
Scattered remnants of kr terraces
Coal Creek
Raleigh Creek
Sawyers Creek
Mill Creek
Omakemoko Creek
Hill Creek
Racecourse Creek
Grandfather Creek
Ernest Creek
Martin Creek
S.Lat.42°25'
S.Lat.42°30'
E.Long.171°10'
E.Long.171°15

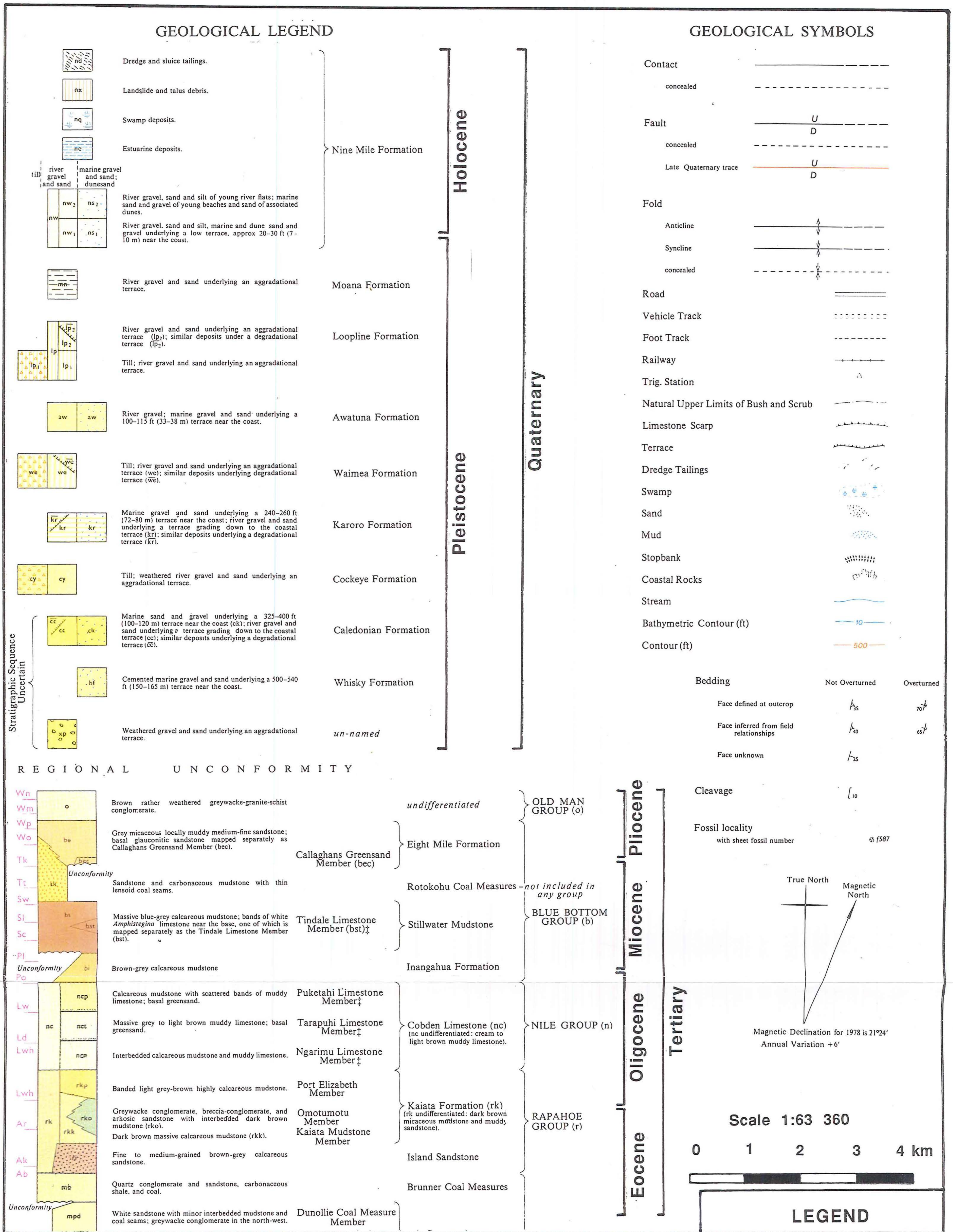


FIGURE 2.1 GREYMOUTH GEOLOGY. From Nathan (1978) NZGS Sheet S44 Greymouth 1978.

2.2.1 BEDROCK.

1. Kaiata Formation (Port Elizabeth Member).

The term Kaiata Formation was proposed by Nathan (1974) to include all brown, micaceous, muddy sediments between the Brunner Coal Measures and the Cobden Limestone. Generally a massive, faintly bedded unit over much of the region, the Kaiata Formation has been subdivided into three members around Greymouth (Figure 2.1) although only the Port Elizabeth Member is present towards the extreme east of the field area. The Port Elizabeth Member consists of light brown, moderately calcareous mudstone interbedded with occasional bands of muddy limestone, and grades over an interval of 3m into the overlying Cobden Limestone.

2. Nile Group (Cobden Limestone).

Nathan (1974) used the term Nile Group to include the various Oligocene limestones along the West Coast. The Nile Group is locally represented by the Cobden Limestone (McKay 1877) and forms a prominent dip slope (including the Twelve Apostles Range and Peter Ridge) defining the western flank of the Brunner-Mt Davy Anticline which confines the field area in the east (Figure 2.1). Although three members (Ngarimu Limestone, Tarapuhi Limestone, and Puketahi Mudstone Members) are recognised by Nathan (1978), the Cobden Limestone generally consists of hard muddy limestone interbedded with calcareous limestone dipping to the west.

Bedding is defined by planar sedimentary layering in beds 0.2-0.6m thick. However, both strike and dip vary along the length of the study area as a result of curvature in the limb of the Brunner-Mt Davy Anticline. Near Point Elizabeth, the dip is around 35° decreasing to less than 20° in the south of the field area (Figure 2.1). The limestone is well jointed and a range of defect orientations are present. The combination of bedding and defect attitudes typically controls the outcrop and landform development, and both of these (bedding and defects) have important implications for the stability of slopes formed within the limestone.

3. Blue Bottom Group (Stillwater Mudstone)

Stillwater Mudstone (Nathan 1974) unconformably overlies the Cobden Limestone and consists of massive, blue-grey, calcareous mudstone of upper Miocene age (Figure 2.1). Bedding within the mudstone is difficult to detect and in the main the unit is featureless. However, the presence of Cobden Limestone boulders and contorted bedding, for example in the Alexandar Street cutting (Sheet S44, grid reference 738874), indicates that slump deposits are interbedded within the massive mudstone (Nathan 1978).

Stillwater Mudstone is the most widespread of the bedrock units within the field area (Figure 2.1). Nathan (1978) indicates that it is probable that the mudstone underlies (at some depth) the coastal plain, and auger hole profiling as part of this study encountered Stillwater Mudstone at depths of 1-3m in some of the major river valleys such as Mill and Sawyers. The soft, relatively uncemented nature of the Stillwater Mudstone has strongly influenced the development of subdued landform morphology, and has led to the formation of areally extensive but relatively shallow regolith deposits that may be susceptible to landsliding under intense or prolonged rainfall.

2.2.2 SURFICIAL GEOLOGY.

Unconsolidated sediments of differing ages and origins have been deposited within the Greymouth area in response to fluctuating sea levels and climates associated with glacial advances and accompanying interglacials, and an uplifting land mass during the Quaternary. The deposits found within the field area may be grouped into the following:

1. Karoro Formation.

The Karoro Formation (Suggate 1965) consists of marine sands and gravels (of greywacke, granitic and schistose origin) partially cemented by iron pan development, and is preserved south of Point Elizabeth as a discontinuous coastal terrace at 72-80m *a.s.l.* (Figure 2.1). Suggate (1965) correlates the Karoro Formation with the Terangi Interglacial (300 000 years before present (B.P.)). Active folding on the Brunner-Mt Davy Anticline has tilted the terrace deposits forming the Karoro Formation towards the west.

2. Loopline Formation.

Extensive aggradational surfaces consisting of sands and gravels (known as the Loopline Formation) were formed in conjunction with ice advances down the Arnold valley during the last (Otira) glaciation (Nathan 1978). Suggate (1965) recognises two separate advances, Loopline-1 (l_{p1}, older) and Loopline-2 (l_{p2}, younger), which are found towards the south east of the study area (Figure 2.1) (Suggate 1965, Suggate and Moar 1970). Radiocarbon dates indicate that the Loopline-2 advance began before 22 000 years B.P. and culminated shortly after 18 000 years B.P.

3. Nine Mile Formation.

Nathan (1978) includes all sediments deposited since the glaciers started to retreat (*c.* 14 000 years B.P.) in the Nine Mile Formation (Figure 2.1). Marine sediments include gravels of greywacke and granitic origin, while fluvial sediments include mud, sand and gravel derived from the breakdown of Cobden Limestone, Stillwater Mudstone and the erosion of the Quaternary deposits. A raised surface (6-9m *a.s.l*) running the length of the field area represents an older part of the Nine Mile Formation (see Figure 2.1) which has been raised to its present height by tectonic uplift (Nathan 1978). The older surface has been radiocarbon dated at 4720+ 70 years B.P. (Suggate 1968). The clear distinction between the two sections of the formation suggests that uplift occurred suddenly, probably following a major earthquake (Nathan 1978).

4. Pedological soils.

Based on Mew (1980), 10 pedological soil types can be recognised within the field area. The distribution (Figure 2.2) and composition (Table 2.1) of these soils are functions of the areal extent and composition of the underlying parent materials.

Karoro and Mahinapua Soils have developed on the younger and older sediments respectively of the Nine Mile Formation, while Kumara soils have formed on the deposits forming the Karoro Formation. Rotokohu soils have formed on the outwash gravels of the Loopline Formation, and Kamaka Soils are present on the low angle (<5°) fans built from colluvium and alluvium derived from Stillwater Mudstone and Cobden Limestone.

Soils formed from the *in situ* weathering of Stillwater Mudstone (Runanga and Stillwater Hill Soils) and Cobden Limestone (Omoto Steepland and Kaiata Hill Soils) commonly become destabilised under changed conditions such as prolonged rainfall or removal of the vegetation. Engineering geological descriptions and properties of these hill soils are presented in Chapter 3.

5. Colluvium.

Colluvium results from the weathering of parent material and is transported downslope mainly through the action of gravity. Within the field area, localised colluvium deposits are confined to the coastal escarpment and other slopes formed in Stillwater Mudstone. Compositionally, these deposits consist of a silty-clay, matrix supported material (Stillwater and Runanga Hill Soils) and incorporate sands and gravels eroded from the Karoro Formation and highly weathered clasts of Stillwater Mudstone. Clasts exceeding 150mm in size are rare and are usually well rounded. Profile thicknesses are not great, reflecting the dominant influence of mass movement processes within the field area and generally range from approximately 1.5m at the slope crest to about 3m at the slope toe.

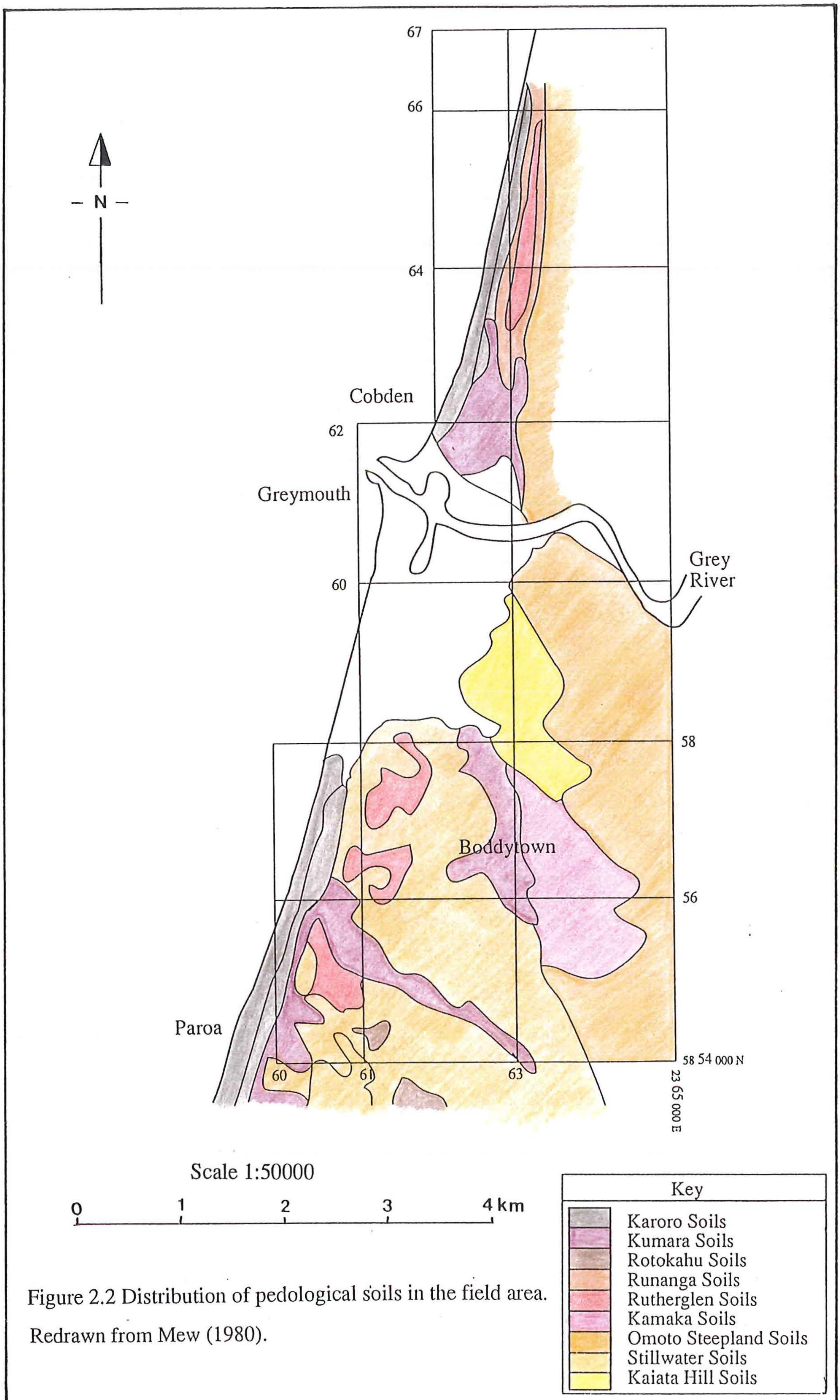


Figure 2.2 Distribution of pedological soils in the field area.
Redrawn from Mew (1980).

Table 2.1 Abbreviated soil descriptions from Mew (1980).

Soil type	Parent material	Vegetation	Slope
Karoro	Dune sand (granite, greywacke and some schist)	Pasture and scrub patches	6-10°
Mahinapua	Dune sand (granite, greywacke and schist)	Mostly pasture, some forest remanents and some scrub	0-22°
Kumara	Loess and some glacial outwash (granite, greywacke and schist)	Cutover-virgin and exotic forest	Flat terraces
Rotokohu	Peat and some alluvium	Cut-over forest and scrub	Flat
Rutherglen	Marine sands, some gravel and loess.	Swamp species	<5°
Kamaka	Colluvium and alluvium	Some pasture and exotic forestry	<5°
Kaiata Hill	Cobden Limestone with related slope deposits	Poor pasture and scrub	>20°
Runanga Hill	Stillwater Mudstone	Cut-over to virgin forest	>20°
Stillwater Hill	Stillwater Mudstone	Cutover forest and pasture grasses	>20°
Omoto Steepland	Cobden Limestone	Cut-over to virgin forest.	>20°

6. Landslide deposits.

Landslides (discussed further in Chapters 4 and 5) are common within the field area and these range from small surficial failures to major slides involving bedrock. In general, shallow surficial landslide deposits are too small to be mapped at the 1:10 000 scale used in this study. However, some of the larger landslides such as rock avalanches have created localised areas of blocky limestone deposits, whilst smaller landslides including slumps or regolith failures involving soils or colluvium have formed small areas of landslide debris.

2.3 STRUCTURE AND TECTONICS.

The Alpine Fault zone (discussed briefly in Section 1.4) is the dominant structural element within the region (Wellman and Willett 1942). The active compressional element of tectonic strain along the fault zone is responsible for the high uplift rates which have created the Southern Alps. Values for the uplift rate of the Alps are 5 mm/year at Okuru (Cooper and Bishop 1979), 8 mm/year at Paringa (Suggate 1968), 20 mm/year adjacent to the highest part of the Alps (Wellman 1979), and 22 mm/year at the Wanganui River (Adams 1978, 1980).

Greymouth lies on the western limb of the Brunner-Mt Davy Anticline and the associated Grey Valley Syncline complex (Nathan 1978). These two folds share a common steeply dipping limb and are highly asymmetric in profile (Dibble and Suggate 1956). Both of these structures have been active since at least the Miocene and are included in the Paparoa Tectonic Zone (PTZ) by Laird (1968).

The Paparoa Tectonic Zone is a broad shear zone extending 190 km from the Alpine Fault at the Hokitika River to Kongahu Point (West Nelson) and it incorporates NNE trending faults and folds (see Figure 1.3). Activity along structural elements within the PTZ, includes faulting (Montgomerie; Hawera; White Creek and the Roa - Mt Buckley Fault Zones) and folding (Brunner-Mt Davy Anticline; Grey Valley Syncline and others), and is responsible for deformation of Quaternary - recent sediments (Suggate 1985, 1987, 1988, Nathan *et al.* 1986, and Nathan 1978).

Off-shore oil exploration indicates the presence of a highly complex Cretaceous fault zone trending roughly parallel to the PTZ (Bishop 1992). The eastern edge of this fault zone (known as the Cape Foulwind Fault Zone) consists of the Cape Foulwind Fault (Figure 2.1), which is an extension of a major fault in the north (Esso 1969, Haematite Petroleum (NZ) Ltd. 1970). Uplift along the Cape Foulwind Fault is thought to be responsible for the present day position of the Westland coast (Nathan 1978).

2.4 GEOMORPHOLOGICAL EVOLUTION.

2.4.1 TECTONIC ACTIVITY.

Greymouth has experienced significant effects from earthquakes during historic times, in particular the 1929 Murchison (M = 5.5-6) and the 1968 Inangahua (M = 7.1) earthquakes. During the former, 60 000-100 000 tonnes of rock fell in the Cobden Quarry and were thrown nearly 100m (Greymouth Evening Star 17/06/1929), whilst liquefaction of coastal sediments around the Blaketown Lagoon occurred during the latter (Berrill *et al.* 1988, Fairless and Berrill 1984). Benn (1992) has compiled an historical account of earthquakes to have affected the Greymouth area. Damage during most of these documented earthquakes includes dislodged chimneys and broken crockery, although landsliding at Omoto occurred during the May 1962 Westport earthquake (M = 5.9) and extensive slipping occurred during the February 1867 earthquake (Grey River Argus (GRA) 07/02/1867).

Greymouth lies adjacent to the paleoseismic gap on the Alpine Fault. Adams (1980) suggests a recurrence interval for earthquakes along this section of the Alpine Fault of approximately 500 years with M = 8.0. The last earthquake (as interpreted from the geological record) was *c.* 550 years B.P. Greymouth could expect significant landsliding including rock falls, avalanches and the initiation of regolith failures following such an event.

Suggate (1968) stated that a major earthquake probably caused the formation of the low scarp separating the younger and older phases of the Nine Mile Formation. Although no fault traces are present within the field area, several large faults present outside the field area could be responsible for such an event. The most likely of these is the Cape Foulwind Fault which Nathan *et al.* (1986) believe responsible for controlling the present day position of the coast.

There have been no earthquakes centred within the Greymouth district in historic times (Nathan 1978). However, given the damage that was sustained during the earthquakes outlined above and the proximity to the active Alpine and Cape Foulwind Faults, then the initiation of slope movements within the Greymouth area is highly probable following a major event. Smith (1990) calculated return periods (years) for various earthquake intensities that can be expected at Greymouth as follows MM = 6 every 10 years, MM = 7 every 34 years, MM = 8 every 110 years, and MM = 9 every 410 years.

2.4.2 SLOPE DEVELOPMENT.

Slopes within the study area range from gentle (<10°) to very steep (>50°) and have formed by four processes:

1) active growth of the Brunner-Mt Davy Anticline;
2) marine cliffing during the Holocene;
3) stream incision; and
4) active tectonic uplift.

Slopes along the Twelve Apostles Range and Peter Ridge (see Figure 1.2) have formed in conjunction with the active development of the Brunner-Mt Davy Anticline. The limestone strata forming the western limb of this large structure dip to the west and are subparallel to the steep slopes (20°+). Numerous back scarps (see Figure 2.1) along the length of these two ranges attest to the fact that much of the original thickness of the limestone, and certainly most of the overlying mudstone, has been removed by erosion. Suggate and Moar (1970) inferred from pollen analysis that a bleak treeless landscape existed during the Loopline-2 advance (22 000-18 000 years B.P.). Therefore mass wasting processes almost certainly dominated during this time and the implication is that present slopes (although subsequently modified) date from Loopline-2 times.

During the Quaternary, continuing regional uplift was superimposed on the pattern of changing climate and sea level. As a result, interglacial terraces (including the Karoro Formation) were raised above the maximum sea level of the succeeding interglacial period and were thus preserved (Nathan 1978). As sea level fell relative to the land, coastal processes created the coastal escarpment cut in Stillwater Mudstone that can be traced to the southern limit of the field area. Nathan (1978) suggests an age for the older part of the Nine Mile Formation, which abuts against the toe of the escarpment, of *c.* 5000-4000 years B.P. Therefore the age of the cliff is constrained to around this time, and several large bedrock failures that have been identified in this study along the cliff probably date from the age of cliff formation (5000-4000 years B.P.). Other landslides involving regolith material must post-date the formation of the cliff.

Valleys formed in Cobden Limestone have generally incised along an E/W trending joint set. Slope angles on the valley sides are very steep (30°+) and in many places are near vertical.

Sea level was at approximately its current height during the Terangi interglacial (300 000 years B.P.). Nathan (1978) infers from the distribution of Karoro Formation gravels within the Grey Valley, that the Grey River almost certainly flowed out to sea through its present course during this time (*c.* 300 000 years B.P.). Sea level fluctuated during succeeding glacial/interglacials and then fell to a low level during the deposition of the Loopline-1 advance (Otira last glaciation, which post dates deposition of the Karoro Formation). Nathan (1978) believes that during the deposition of the Loopline-1 advance, the sea level was "much" lower than at present and the coastline was further to the west. Therefore the subvertical to vertical limestone cliffs around the Grey River Gorge and other river valley slopes formed within Stillwater Mudstone (i.e. Sawyers and Mill creeks etc.) have probably formed as sea level fell during the deposition of the Loopline-1 advance. The presence of 53m of gravels (detected during foundation drilling for the Greymouth-Cobden bridge) at the Grey River Gorge suggests that rivers have generally aggraded since Loopline-1 times.

2.4.3 WEATHERING PROCESSES.

Stillwater Mudstone and Cobden Limestone differ markedly in both the type of weathering that occurs and in the depth of alteration. Regolith development on Cobden Limestone is generally less than 2m deep, and occurs primarily by chemical alteration of the limestone surface and solutioning of the calcite cement. Joint faces within the limestone are iron stained yellow-brown in colour, indicating the importance of defects in allowing water into the rock mass. Sink holes often result, the depth and magnitude of which cannot be determined from surface observation.

Weathering of Stillwater Mudstone (Figure 2.3) may extend to greater depths (≤4m) than within Cobden Limestone and is of particular significance in relation to slope stability. Jointing appears to allow ground water into the rock mass generating high pore pressures leading to weakening and disintegration of the rock material by slaking. The resulting *in situ* or colluvial products commonly become destabilised under changing conditions such as removal of the vegetation cover, during high rainfall or in cut slopes accompanying urban development.

Figure 2.3 Typical exposure and weathering of the Stillwater Mudstone. Note highly fractured nature of the outcrop and lack of defect continunity. Defects are zones of alteration within the rock mass.

2.5 SLOPE STABILITY IMPLICATIONS.

2.5.1 BEDROCK.

Along the Twelve Apostles Range and Peter Ridge (hard rock), failure may be expected of the tectonically oversteepened weaker, mud-rich, planar sedimentary beds "daylighting" on slopes with defect controlled release along lateral and head scarp margins. In particular, where ground water infiltrating the rock material through rock mass features and sink holes has created weakened mud rich layers and is contributing to increased pore pressures. In those areas of very steep terrain, such as the incised river valleys or around the Grey River Gorge, the defect orientations may contribute to fall or topple type failures, and may include a component of sliding along steeply dipping defects.

Areally extensive, deep seated translational landslides, sliding along low shear strength dipping as low as 5° are common in the soft rock terrain of New Zealand (Bell and Pettinga 1988, Brown 1974). Therefore similar landslides can be expected in Stillwater Mudstone failing along incipent bedding surfaces dipping around 20-25° which "daylight" on cut or natural slopes.

2.5.2 SURFICIAL MATERIALS.

Stillwater Mudstone weathers to produce fine grained, cohesive regolith deposits, as does Cobden Limestone although much less rapidly. These deposits, and including associated colluvial products, can be expected to fail under intense or prolonged rainfall assisted by high pore pressures along the bedrock/ soil or colluvium interface, producing shallow regolith landslides and mudflows.

2.5.3 CAUSATIVE FACTORS.

Inherent (pre-existing) factors present within the Greymouth field area that may contribute to mass movements include: site geology; hydrogeology and topography; whilst initiating (triggering) factors within the Greymouth field area that could be important in causing landslides are earthquakes, intense rainstorms, deforestation and urban development.

Earthquake-generated failures could include slides, falls and rock avalanches (regarded as one of the most hazardous landslide types). Rainstorm generated failures could include both translational and rotational slides, debris flows or a combination of these. Surficial failures commonly impact on dwellings or occur in cut slopes, for example roading batters. Extensive logging of the natural forest cover and the impact of urban development may also be important factors in landslide development.

2.6 SUMMARY.

The field area is underlain by Eocene-recent sedimentary rocks forming the western flank of the Brunner-Mt Davy Anticline and lies to the east of the Cape Foulwind Fault Zone. Bedrock within the mapped area consists of Eocene mudstone, Oligocene limestones and Miocene mudstone. These are unconformably overlain by unconsolidated sands and gravels preserved on marine terraces, tectonically uplifted during the Quaternary. Latest Holocene weathering of bedrock and surficial map units has created *in situ* regolith and transported colluvial deposits. Weathering profile thicknesses on the steep hill country (Twelve Apostles Range and Peter Ridge), river valley sides and coastal escarpment are generally less than 3m deep.

Surficial and bedrock materials within the mapped area have the potential to be susceptible to mass movements caused by both inherent (pre-existing) and initiating (triggering) factors. Variables included within the former category include; site geology, hydrogeology, and topography while factors included within the latter category include earthquakes, intense rainstorms, deforestation and urban development. Landslides in the Greymouth area have important implications for both existing land use and future residential development.

CHAPTER 3.
ENGINEERING GEOLOGICAL INVESTIGATIONS.

3.1 INTRODUCTION.

The engineering geological field investigations, and the results of the rock and soil characterisation carried out as part of this study are presented in this chapter. The primary aim of the field investigation was to obtain information that was relevant to a landslide hazard zonation. The conceptual stages (Figure 3.1) involved background studies including aerial photograph interpretation, field investigation and laboratory testing. The objectives of the investigation were as follows:

1) to compile an engineering geological map of the study area at a scale of 1:10 000; and

2) to conduct geotechnical characterisation of representative rock and soil materials.

The investigation methodology is detailed in Section 3.2 and the results of the rock material and rock mass characterisation are presented in Section 3.3 and 3.4 respectively. The data obtained during soil characterisation is presented in Section 3.5.

3.2 INVESTIGATION METHODOLOGY.

3.2.1 AERIAL PHOTOGRAPHIC INTERPRETATION.

Sets of aerial photographs (Table 3.1) providing full coverage of the study area were obtained from various sources. Interpretation of these photographs formed the basis of the engineering geological mapping, and the photographs proved essential aids in the identification of slope movements. The importance of obtaining aerial photographs at a range of scales was realised, since while it was possible to identify the larger features on the smaller scale photographs, the larger scale photographs were necessary for the recognition of small landslide features.

Fresh landslide scars were clearly discernible on aerial photographs, even on heavily vegetated slopes. Brabb (1984) recognised that with time, however, a landslide scar may become unrecognisable either through degradation or erosion of its features, or through the regeneration of vegetation. This was particularly true in the Greymouth setting where high erosion and vegetation growth rates result from a high annual rainfall. In order to overcome the problem of recognising older landslides, where landslide morphology degradation had occurred, it was necessary to obtain aerial photographs that had been taken at different times over the last fifty years (the length of time representing aerial photograph coverage refer Table 3.1). Thus it was also possible to assign relative ages to landslides that were absent on earlier aerial photographs but were present on the later aerial photographs. The age of identified landslides was then used in the development of a landslide hazard zonation of the area (discussed further in Chapter 5).

Landslide features identified on the aerial photographs were traced onto an overlay which was reduced or enlarged to the scale of 1:10 000, and then retraced onto base maps. Field checking of features identified from the photographs was then undertaken where-ever possible.

3.2.2 ENGINEERING GEOLOGICAL MAPPING.

Field mapping had two main objectives: 1) to delineate spatial geologic relationships; and 2) to establish a landslide inventory for the purpose of hazard zonation. The former was based on the stratigraphic subdivisions used by Nathan (1978) (see Figure 2.1), while the latter necessitated the development of a landslide inventory sheet for data recording in the field (Figure 3.2). This was based largely on the terminology of Varnes (1978), as outlined in Chapter 1.

The engineering geological field data and including information obtained by aerial photographic interpretation is given in Figures 3.3a, b, c (in the map pocket at the back). These represent an engineering geological data base from which landslide hazard zonation and land use capability maps have been derived (Chapter 5). Base maps used in the field investigation were compiled from the photographic enlargement of a 1:25 000 topographical map from the Department of Survey and Land Information. Although the photographic enlargements were supposedly distortion free, the original 1:25 000 map is stored on micro film and in production, significant distortion was apparent, particularly around the margins of the maps. This was overcome by overlying adjacent sections until a satisfactory match could be obtained. The enlargements were then traced to form the three maps that provide full coverage of the field area.

In areas of dense vegetation, it was possible to infer landslide features on the basis of localised changes in the vegetation cover (Figure 3.4). In general, however, the vegetation (especially gorse and blackberry) hindered field investigation for two reasons. Firstly, the dense nature of the undergrowth, particularly in areas of regenerating forest, prevented site access and secondly, the vegetation obscured landslide morphology. It is therefore likely that some landslides have still not been identified during field mapping.

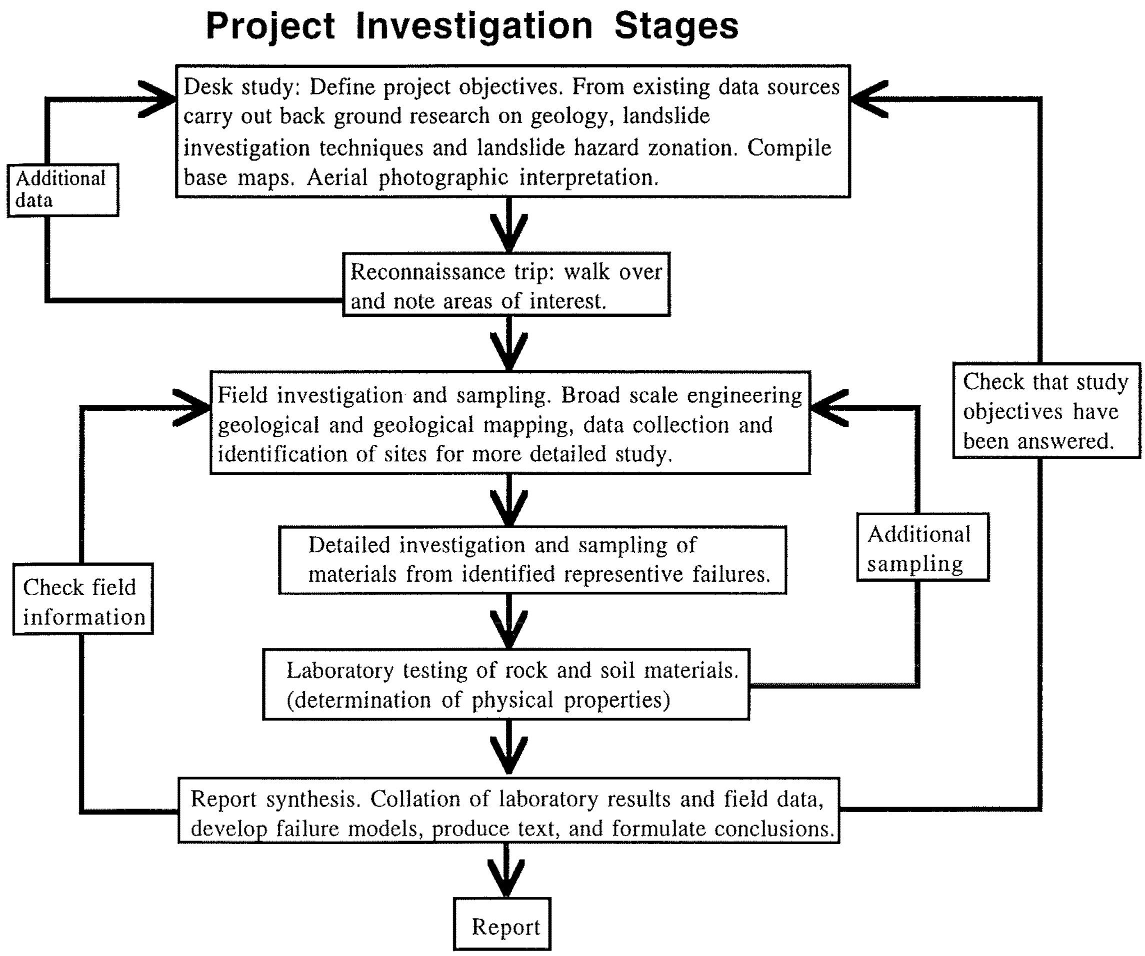


Figure 3.1 Flow chart illustrating the methodology involved in the project. The conceptual stages were adopted from Clayton *et al.* (1982).

Table 3.1 List of aerial photographs used in the course of this study.

Run Number	Numbers Used	Scale	Date Taken	Source
811 813 814 815 816	1 -4 1 - 7 1 - 7 1 - 6 1 - 6	1:16 000	1943	Geology Dept.
5782	D5 - D10 E5 - E16 F8 - 14	1:10 000	1986	DOSLI
SN C8922	Z/7 - Z/15	1:15 000	1988	DOC
Untilted Untilted		1:10 000 1:5000	1993	TWCRC

Note: DOSLI = Departmant of Survey and Land Information.
DOC = Department of Conservation.
TWCRC = The West Coast Regional Council.

MASS MOVEMENT INVENTORY FORM

failure reference	
map reference - sheet no.	
coordinate ref	
brief location	
date of failure (if known)	
date of observation	

GEOLOGY

Main rock type	
Main soil type	
Debris type	

DIMENSIONS OF FAILURE

width		length	
areal extent		depth of failure	

SURFACE FEATURES

fresh		partly veg	
vegetated		hardly visible	

slope	
main triggering factor	

bedding	
defects	

SURFACE HYDROGEOLOGY

dry		partly wet	
large areas wet		springs, present	

PROBABILITY OF FURTHER MOVEMENTS

very low		medium	
low		high	

ACTIVITY OF FAILURE

active	inactive	partly active

TYPE OF MATERIAL BEFORE MOVEMENT

rock		soil		debris	

TYPE OF FAILURE

fall		topple	
complex		flow	
slide		translational	
		rotational	

Comment

Photo (10*15 cm)/Sketch

comment

Figure 3.2

Figure 3.4 A landslide near Boddytown that was identified from localised and distinct changes in vegetation patterns.

3.2.3 FIELD INVESTIGATION AND SAMPLING.

Exposure logging of outcrops and of shallow (<2m deep) hand dug pits constituted the principal source of information on rock and soil materials. Useful qualitative information on rock mass relationships and the influence of structure on rock mass stability was obtained through mapping (at a scale of 1:100) small cliffs cut in Cobden Limestone around the Cobden Bridge. Information on the structure of regolith and colluvial deposits was obtained through face logs (at a scale of 1:20) of shallow hand dug pits.

Engineering geological descriptions of the various rocks and soils were recorded in the field and photographs were taken for future reference. Bulk samples of selected rock and soil lithologies were collected for laboratory analysis including:

1) point load strength testing;
2) porosity and density determinations;
3) slake durability testing;
4) particle size analysis;
5) natural soil moisture content determination;
6) Atterberg Limits and related indices; and
7) clay mineralogy.

The purpose of laboratory testing was to determine physical and mechanical rock and soil properties that would provide an indication as to the stability and performance of these materials in the field.

Hand auger holes (diameter 50mm) were used to acquire information on the stratigraphy and nature of slide debris, ground water and soil materials. Profiles were logged from disturbed cuttings and some samples were collected for laboratory testing. In addition, the auger holes were used to provide depth correlation for a seismic refraction line that was carried out in the investigation of the Australasian Hotel Slide this having been identified during field mapping.

Several defect scan lines were carried out along sections of the Grey River Gorge. These were used to obtain quantitative data on the defects within the Cobden Limestone that may contribute to the instability of this rock unit. The results of these are discussed in section 3.4.

3.3 ROCK MATERIAL CHARACTERISATION.

3.3.1 DESCRIPTIONS OF LITHOLOGIES.

Five rock lithotypes were recognised in the field (based on Nathan 1978), and engineering geological descriptions of these are given in Table 3.2.

Table 3.2 Rock material field descriptions.

Lithotype	Weathering term	Strength term	Colour term	Structure term	Rock name
Port Elizabeth Member Lithotype 1.	Slightly	Moderately weak	Dark brown-grey	massive	Calcareous MUDSTONE
Nagrimu Limestone Member Lithotype 2a.	Slightly	Extremely strong	Light brownish-grey	massive	MUDDY LIMESTONE
Lithotype 2b.	Slightly	Moderately strong	Dark brownish-grey	massive	Calcareous MUDSTONE
Tarapuhi Limestone Member Lithotype 3.	Unweathered	Extremely strong	Light brownish-grey	massive	MUDDY LIMESTONE
Puketahi Limestone Member Lithotype 4a.	Slightly-moderately	Very strong	Light bluish-grey	massive	MUDDY LIMESTONE
Lithotype 4b.	Slightly-moderately	Moderately weak	Light bluish-grey	massive	Calcareous MUDSTONE
Stillwater Mudstone Lithotype5a.	Unweathered	Moderately weak	Dark bluish-grey	massive	Calcareous MUDSTONE

Field descriptions after Bell and Pettinga (1983), see Appendix A1.5.

Field descriptions (Table 3.2) indicated that the Tarapuhi Limestone Member (Lithotype 3) consisted of muddy limestone, and that the Stillwater Mudstone (Lithotype 5) and the Port Elizabeth Member (Lithotype 1) consisted of calcareous mudstone. The Ngarimu Limestone and the Puketahi Mudstone Members consisted of alternating muddy limestone (Lithotypes 2a and 4a respectively) and calcareous mudstone (Lithotypes 2b and 4b respectively). Samples of these lithotypes (excluding Port Elizabeth Member) were collected for laboratory testing. No geotechnical testing was undertaken for the Port Elizabeth member owing 1) to the extremely localised occurrence of the mudstone in the field area; and 2) to the difficulty of collecting suitable samples.

3.3.2 PHYSICAL PROPERTIES.

3.3.2.1 Porosity and density.

Porosity and density determinations for the lithotypes involved six sets of ten irregular shaped samples and utilised the saturation and buoyancy techniques suggested by Brown (1981). The method involves the immersion and saturation of irregular shaped specimens and allows the following properties to be determined:

Dry Density (ρ_d) = The mass of the grains per total unit volume, that is solids and voids with the voids empty.

Porosity (n) = The volume of voids per total unit volume.

The porosity of a rock is defined as "the ratio of the volume of voids in a rock to its total volume" (Jumikis 1983) and is dependent upon the type and structure of the rock. Description of the pores in a rock is important since a small pore volume can produce an appreciable mechanical effect through a decrease in strength and an increase in deformability (Brown 1981). The density of a rock is defined as the "mass per unit volume" (Lama and Vutukri 1978).

Table 3.3 Summary of porosity and density determinations.

Lithotype	Number of samples	n (%)	ρd (t/m3)
Lithotype 2a	10	6.77	2.5
Lithotype 2b	10	19	1.9
Lithotype 3	10	6.78	2.6
Lithotype 4a	10	3.33	2.6
Lithotype 4b	10	17	2.4
Lithotype 5	10	38	1.8

The high porosity values (Table 3.3) for the mud-rich lithotypes (2b, 4b and 5) arise from the large void volume that is inherent within rocks of predominantly fine particle sizes and was a function of the large numbers of small fractures that developed during sample desiccation on exposure to air. Low porosity values obtained for lithotypes 2a, 3 and 4a were attributed to the higher calcium carbonate content that these rocks are expected to have. The permeability of *in situ* rock material (primary permeability) of both the Stillwater Mudstone and the Cobden Limestone was expected to be low, although the secondary permeability, via rock mass defects, was probably very high.

Density values range from 1.77 t/m^3 (Lithotype 5) to 2.6 t/m^3 (Lithotype 3 and 4a). The density values are clustered around 2.56t/m^3.

3.3.2.2 Slake durability testing.

Those rocks containing a high clay content are prone to swelling, weakening or disintegration when subjected to short term weathering processes (Brown 1981). Slake durability testing is intended to assess the resistance offered by a rock sample to weakening and disintegration when subjected to two cycles of wetting and drying (Vutukri 1978). The nature of the test method means that abrasion will also contribute to the disintegration of the sample.

Field descriptions (Section 3.3.1) indicated that the rock lithotypes in the field contained an appreciable clay content. Hence it was felt that testing these rocks for resistance to weathering was particularly relevant. Testing followed the methodology of Brown (1981) and involved six sets of ten irregular shaped samples. The slake durability test is classified on the basis of the second cycle (I_{d2}) although the lithotypes were subjected to four slaking cycles (I_{d4}). The results are summarised in Table 3.4 and illustrated in Figure 3.5.

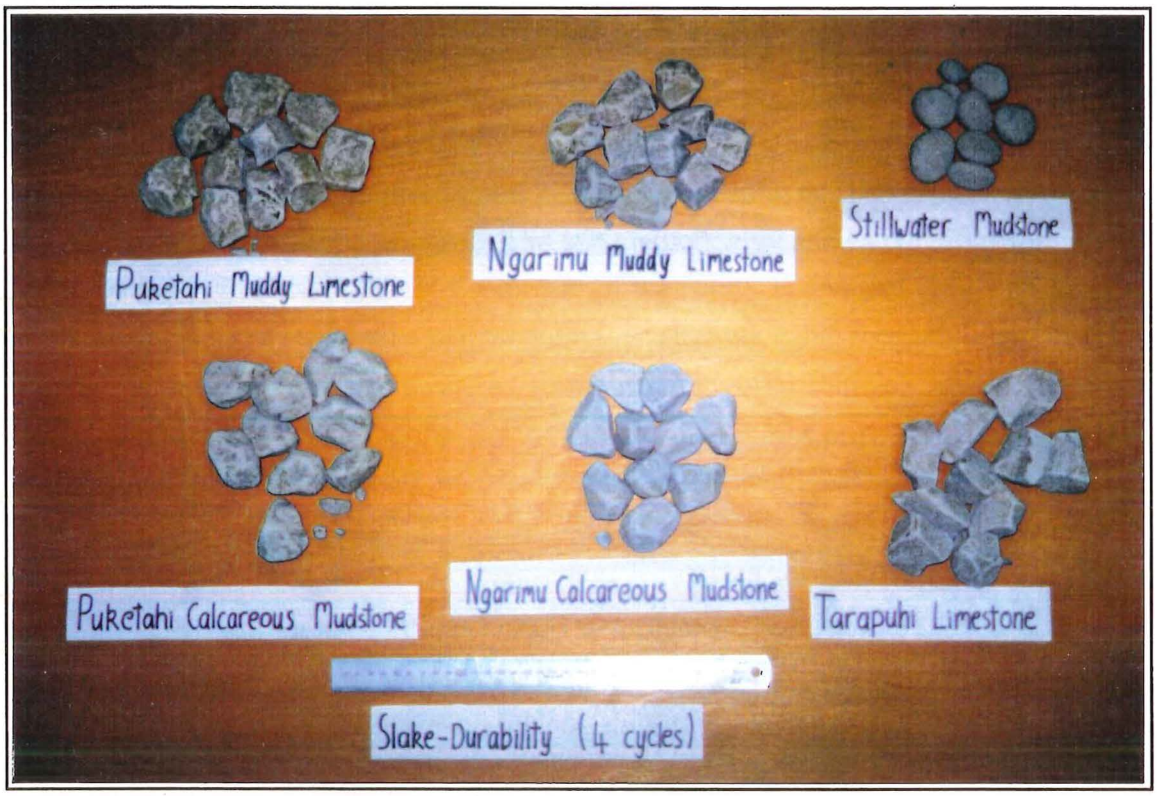


Figure 3.5 The Lithotype samples after four slaking cycles. Note the highly rounded nature of the Stillwater samples in contrast to the various lithotypes of the Cobden Limestone.

Table 3.4 Summary of slake durability testing.

Lithotype	Number of samples	Id2 (%)	Id4 (%)
Lithotype 2a	10	97.5	96.1
Lithotype 2b	10	96.1	93.0
Lithotype 3	10	99.1	99.1
Lithotype 4a	10	97.6	96.2
Lithotype 4b	10	95.0	91.4
Lithotype 5	10	45.1	18.8

Classification of the slake durability data after Gamble (1971) revealed that the various lithotypes of the Cobden Limestone (see Figure 3.5) were included in the high resistance to erosion category. Even after four slaking cycles (I_{d4} in Table 3.4 above) the lithotypes were included in or above the medium resistance to erosion category (after Gamble 1971). Stillwater Mudstone displayed little resistance to erosion (see Figure 3.5) particularly after four slaking cycles.

3.3.3 MECHANICAL PROPERTIES.

3.3.3.1 Point load strength.

Rock strength determinations involved point load strength testing on six sets of twelve irregular shaped representative lithotype samples. The test methodology and calculations followed the suggested method of ISRM (1984). A summary of the results is given in Table 3.5. Each set of results were averaged to obtain a unique point load strength value for each lithotype ($Is_{(50)}$ (average)). Although the test is designed to be used on cored samples with a diameter of 50mm, it can never-the-less be applied to irregular shaped lumps providing that a correction factor is applied.

The point load strength test may be empirically correlated with unconfined compressive strength. However, as Bell and Pettinga (1983) have stated:

> *"we reject the conversion of point load strength values to equivalent unconfined compressive strength because of serious doubts as to the validity of the empirical correlations quoted (for example, by Broch and Franklin, 1972). The point load strength should be used as an index in its own right."*

For this reason, the point load strength values have not been correlated with the unconfined compressive strength.

Table 3.5 Summary results of the point load strength test.

Sample	Number of samples	Average Is(50) (MPa)
Lithotype 2a	12	2.35
Lithotype 2b	12	1.77
Lithotype 3	12	3.1
Lithotype 4a	12	2.78
Lithotype 4b	12	1.6
Lithotype 5	12	0.46

Point load strength values (Table 3.5) ranged from 3.1 (Lithotype 3) to 0.46MPa (Lithotype 5). The lithotypes included in the Cobden Limestone had a much greater strength relative to the Stillwater Mudstone. However it is apparent that the calcareous mudstone units (Lithotypes 2b and 4b) within the limestone had a slightly lower strength than the muddy limestone units (Lithotypes 2a, 4a and 3).

3.3.4 SUMMARY.

Mean data for point load strength testing, porosity and density determinations, and slake durability are illustrated in Table 3.6.

Table 3.6 Bedrock characterisation data summary.

Sample	$I_{s(50)}$ (Average)	I_{d2}	I_{d4}	n	ρd
Lithotype 2a	2.35	97.5	96.1	6.77	2.53
Lithotype 2b	1.77	96.1	93	19	3.65
Lithotype 3	3.1	99.1	98.2	6.78	2.56
Lithotype 4a	2.78	97.6	96.2	3.33	2.56
Lithotype 4b	1.6	95	91.4	17	2.43
Lithotype 5	0.46	45.1	18.8	38	1.77

Testing has shown that the various lithotypes of the Cobden Limestone have in general a relatively high strength (Is (50)) and a medium-very high resistance to erosion (I_{p2} and I_{p4}). The Calcareous Mudstone Lithotypes (2b and 4b) had a slightly lower strength (Is(50)) and resistance to erosion. These differences are in agreement with the differences in strength that were observed in the field (refer Table 3.5).

Stillwater Mudstone has a low strength (Is(50)) and low-very low resistance to erosion (Ip2 and Ip4).

The tests results obtained indicate that the Cobden Limestone should be a suitable foundation material and resist deformation and erosion processes. Stillwater Mudstone appears unsuited as a foundation material with low strength and resistance to erosion.

3.4 ROCK MASS CHARACTERISATION.

3.4.1 FIELD DESCRIPTIONS.

Field investigations ascertained that the Cobden Limestone contains regularly spaced joints (around 1-1.5m) and that these joints (in combination) appear to directly affect the stability of the rock. Qualitative and quantitative (discussed in Section 3.4.2) analysis was undertaken in an attempt to determine the influence of structure on rock stability (Section 3.4.3).

The major defects appeared highly persistent in orientation and apart from minor asperities, had smooth joint surfaces. The wave amplitude for these defects ranged from near 0 to around 0.2m and the wave period was generally in the range 2.0-5.0m. The mud-rich sedimentary beds (Lithotypes 2b and 4b) fractured readily on exposure to air. The continuity of these fractures was generally less than 0.1m and they do not appear to penetrate the rock to any great degree.

A low volume spring (<5*l*/minute) was observed issuing from a cliff on the south side of the Cobden Bridge and most of the major joint surfaces were iron stained yellow-brown in colour, indicating that ground water could have a strong destabilising influence on the rock. Field observation followed a dry period of weather and surface water was absent on most of the defect walls.

Surface and subsurface cavities (sink holes) have developed within the limestone (Figure 3.6). These range in size upwards from 1m in diameter. The depth of the larger cavities could not be determined from field observation. In general, cavities appeared to form along defects and many of the stream channels in the limestone have been entrained by jointing within the limestone.

No gouge was observed in any of the joints and in general the joints were "tight". However, in one locality, significant relaxation of the limestone had taken place (Figure 3.7). In this instance, the joints had been infilled with regolith from above.

In contrast to the Cobden Limestone, the defects within Stillwater Mudstone were highly variable in orientation. The mudstone fractured and slaked rapidly on exposure to air and rapidly formed colluvial and regolith deposits. Defects within the mudstone were zones of alteration which contributed to the poor quality of the rock in general.

Figure 3.6 Small sink hole developed in the Cobden Limestone. This was exposed during foundation excavation for a retirement home in the Puketahi Street area.

Figure 3.7 Joint relaxation in the Cobden Limestone in the Mount Street area. The joints have been infilled with material washed down from above. Note the regular spaced jointing in the limestone.

3.4.2 DEFECT SURVEYS.

Scan lines following the suggested method of Piteau (1971) were carried out along sections of the Grey River Gorge for the purpose of obtaining quantitative information on the defects that may affect rock stability. The strike and dip of defects were recorded and the bedding attitudes were recorded at regular intervals. Because the tape was not always "hard" against the rock face it was often necessary to project the intersection of the defect onto the tape line using a metre rule.

In order to minimise the effects of undersampling due to the orientation of the scan line in relation to the defect, the defect populations that were identified were corrected for true frequency. The analysis (Following Hoek and Bray 1981) of 144 defects sampled from a total scan line of 232m (Figure 3.8) revealed the following defect populations:

F1 = 088/84° N
F2 = 170/16° E
F3 = 174/40° W
F4 = 086/46° N
F5 = 043/74° NW

Bedding: North of the Grey River 173/28° WSW
South of the Grey River 007/27° W

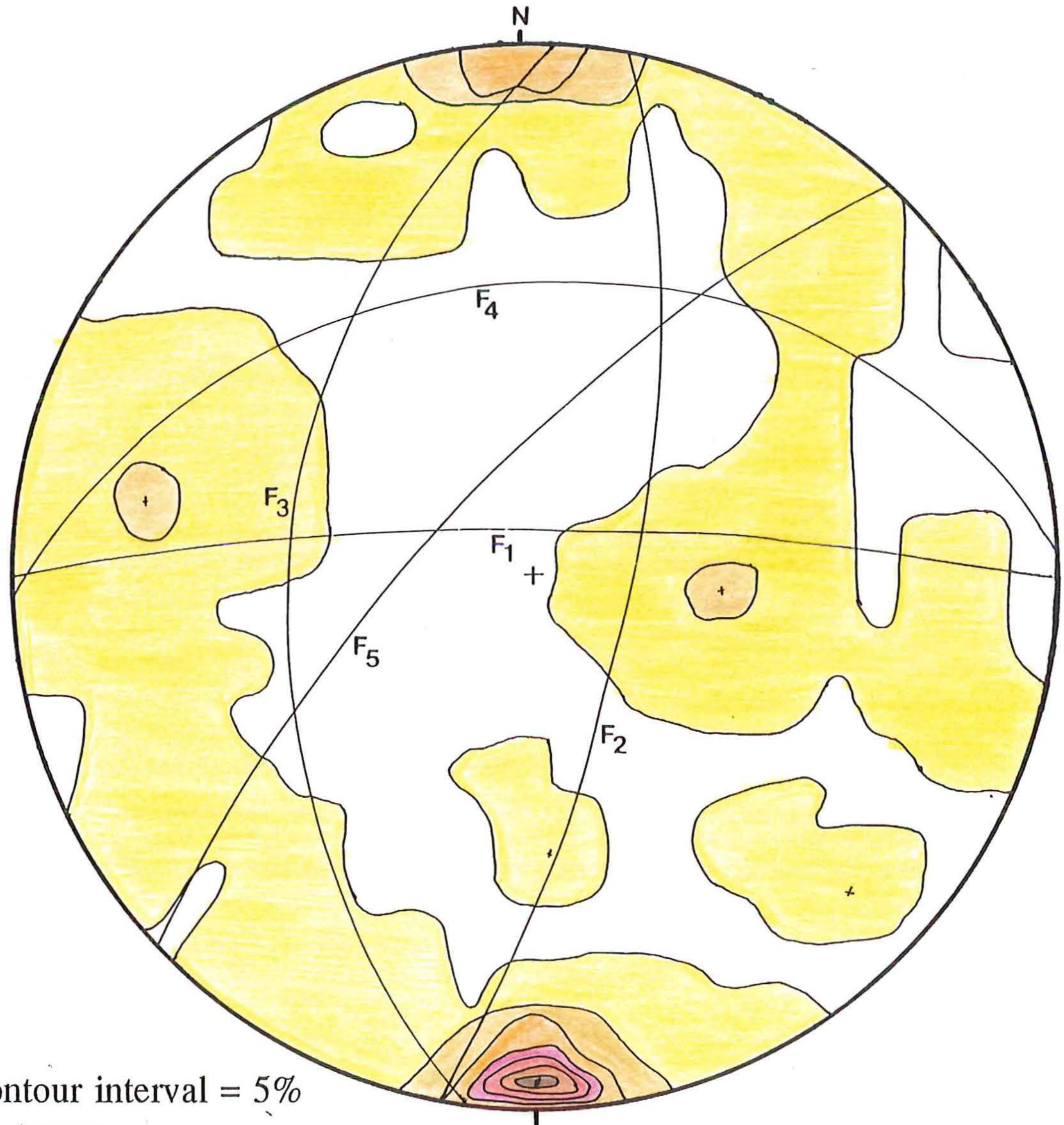
These are regarded as the major defects within the limestone. To assess the validity of the data obtained, it was necessary to statistically analysis the data using confidence intervals. This revealed that defect populations F1, F2 and F3 were both 95 and 99% statistically significant, whereas defect sets F4 and F5 were neither 99 or 95% significant and therefore may just be random occurrences. There are two possible reasons for the latter result, either the defect populations F4 and F5 are anomalous (i.e. random occurrences) or they are very widely spaced and therefore were undersampled. To determine which of these is correct would require a much longer scan line length.

Defects were not measured for the Stillwater Mudstone because of the lack of suitable exposure and poor to absent defect continuity. Defects within the mudstone were highly anastomosing and there was a lack of suitable exposure from which reliable defect orientations could be obtained. Bedding in Stillwater Mudstone was also difficult to measure and in most cases proved impossible.

FIGURE 3.8 CONTOUR PLOT OF POLES TO MAIN DEFECTS IN THE COBDEN LIMESTONE.

Projection = Schmidt.

Number = 144.



Contour interval = 5%

	>30
	25 - 30
	20 - 25
	15 - 20
	10 - 15
	5 - 10
	0 - 5

3.4.3 STABILITY IMPLICATIONS.

Both qualitative and quantitative information on rock mass defects have been obtained for the Cobden Limestone and qualitative information has been obtained for the Stillwater Mudstone.

The major limestone defects (see Section 3.4.2) were assumed to have developed in association with the active growth of the Brunner-Mt Davy Anticline by lateral curvature and flexural folding. However, the absence of bedding plane shears within the limestone would suggest otherwise. Therefore the origin of these defects is unknown. The absence of bedding plane shear indicates that bedding does not appear to be a significant factor in causing instability. However, other defects dipping parallel or subparallel to bedding could act to reduce the stability of the rock.

The range of defects within the limestone is conducive to the development of topple and fall failures, particularly in cut slopes such as foundation cuts or roading batters. Toppling failures releasing on F3 and incorporating a slide component along F2 with lateral releasing on F1 or F4 could occur on east facing batters. Consequentially these four defects could also cause topples on west sloping batters incorporating a slide component along F3 with releasing on F2 and lateral releasing on F1 or F4. The highly persistent and planar defect F1 would allow slab or fall failures on steeply dipping east/west trending slopes (discussed further in Chapter 4).

Weathering and slaking of the mud rich beds within the limestone acts to undermine the competent muddy limestone layers. This could lead to fall failures over time. Sink holes are clearly important in allowing ground water to infiltrate the rock. The stability of the rock is expected to be decreased by pore water pressures from ground water particularly following heavy rain. Water would exert a pressure on joint walls enhancing the likelihood of toppling or fall failures and would also decrease the friction component that is resisting shear along inclined defects.

Stillwater Mudstone contains a range of anastomosing defects. These allow water into the rock and would appear to promote slaking and slabbing type failures. Road batters showed significant decrease in competency over the study period. The rapid loss of strength combined with the highly fractured rock is expected to produce complex rock debris type failures on roading cuts. In addition, high pore pressures developed following heavy or prolonged rainfall is expected to cause translational sliding on low shear strength bedding surfaces daylighting on cut or natural slopes.

3.5 SOIL CHARACTERISATION.

3.5.1 PROFILE DESCRIPTIONS.

Soil profiles were logged (scale 1:20) from pits dug by hand at representative sites. Face logs and full engineering geological descriptions for most of the soil units recognised in the field area (see Figure 2.2) are given in Appendix A2.1. As this study is concerned with slope stability, only those soils present on the slopes (Omoto Steepland, Kaiata, Runanga and Stillwater Hill Soils) are further characterised here. The profile locations are given as a grid reference in the profile descriptions (presented in Appendix A2.1).

Field descriptions ascertained that soils formed *in situ* on the slopes (Omoto Steepland and Kaiata Hill Soils) were fine grained and cohesive, containing an appreciable clay content. A Silty CLAY description was determined for these soils in the field. Runanga and Stillwater soils were present as a fine grained matrix within colluvium formed from the erosion products of the Karoro Formation and Stillwater Mudstone.

Bulk samples of 5 regolith and 5 colluvial matrix materials were collected and sealed within plastic bags (to retain moisture) for laboratory testing. The soil horizons that were sampled are indicated in the soil profile descriptions (Omoto Steepland, Kaiata, Runanga and Stillwater Hill Soils), see Appendix A2.1.

3.5.2 CLASSIFICATION TESTS.

3.5.2.1 Particle size analysis.

Particle size testing followed the method of Lewis (1984). The material that passed through a 4Ø wet sieve was further analysed using hydrometer analysis (Lewis 1984) on the clay fraction. The full results and calculations for the particle size testing is given in Appendix A2.2 and the results are summarised in Table 3.7.

Particle size distribution forms one of the basic criteria used in the engineering geological classification of soils and recognises that the size, shape and composition of soil particles to a large extent dictate the engineering behaviour of that soil. Four descriptive terms are commonly used to describe the particle size (gravel, sand, silt and clay). For the purpose of this study, gravel represents the size fraction coarser than 2mm (-1Ø), the sand fraction ranges between 0.06 - 2mm (4 - $^{-}$1Ø), the silt fraction ranges between 0.002 - 0.06mm (9 - 4Ø) and the clay fraction represents that class of particles finer than 0.002mm (9Ø).

Table 3.7 Summary of particle size analysis.

Sample	Depth (m)	Type	Particle Size		
			Clay	Silt	Sand
Stillwater L3	0.35	c	36	59	5
Stillwater L4	0.35-0.57	c	29	61	10
Stillwater L5	0.57-1.0	c	24	69	7
Omoto L2	0.1-0.26	r	18	72	18
Omoto L3	0.26-0.72	r	34	55	11
Runanga L3	0.53-0.98	c	71	21	8
Runanga L4	0.98-1.43	c	39	38	21
Kaiata L3	0.26-0.43	r	25	60	15
Kaiata L4	0.43-0.83	r	24	62	14
Kaiata L5	0.83-1.23	r	25	65	10

Note: 1) "r" denotes residual soil and "c" denotes colluvial in origin.
2) Soil sample locations are given in the respective profiles.

Particle size analysis (Table 3.7) determined that silt formed the dominant size fraction in almost all of the samples. Clay sized particles comprise the majority of the remaining fraction whilst sand formed a minor constituent, always less than 21 percent and generally less than 15 percent. The determination of a Silty CLAY field description for these samples is therefore valid given the difficulty of distinguishing relative proportions of silts and clays in fine grained cohesive soils in the field.

3.5.2.2 Clay mineralogy.

X-ray diffraction was used to determine the clay mineralogy of the same 10 soil samples and followed the suggested method of Hutchison (1974). X-ray diffraction peaks were obtained using a Phillips X-ray diffractometer with CuK radiation. The resultant graphs and the interpretation (based on Brindley and Brown (1980) of these graphs are presented in Appendix A2.3. The clay minerals identified in the samples are summarised in Table 3.8.

The extent to which the clay mineral fraction dictates the engineering behaviour of the soil is largely dependent upon the water content of the clay. Clay minerals can swell appreciably in the presence of water which can lead to a substantial destabilising effect on the soil (Grim 1962). The kaolinites form the most stable clays as a result of their non-expandable lattice structure. These clays are only moderately plastic when wet and do not incur the destabilising effects of water. The sheets forming the crystal lattice in the montmorillonites are loosely bound and an unstable mineral results. These clays will readily absorb water by both intra and intercrystalline mechanisms. During intracrystalline swelling the lattice sheets are surrounded by water molecules thereby increasing the plasticity and decreasing the internal friction of the soil. Illite clays are similar to the montmorillonites but have a lower capacity for swelling (Beavis 1985).

Table 3.8 Clay mineralogy data summary

Sample	Clay mineralogy
Still L3	Fe Chlorite, Kaolinite, Sepiolite
Still L4	Kaolinite, Fe Chlorite, Sepiolite
Still L5	Kaolinite, Fe Chlorite, Sepiolite, Mica
Omo L1	Fe Chlorite, Kaolinite, Mica
Omo L2	Fe Chlorite, Kaolinite, (Swelling Chlorite or Na Smectite)
Run L3	Swelling Chlorite, Hollysite, Kaolinite, Mica
Run L4	Swelling Chlorite, Fe Chlorite, Mica, Kaolinite, Imogolite
Kai L3	Fe Chlorite, Kaolinite, Na Smectite
Kai L4	Fe Chlorite, Kaolinite, Na Smectite
Kai L5	Fe Chlorite, Kaolinite, Sepiolite, Mica

Samples consisted of Fe Chlorite, Kaolinite, Mica, Sepiolite, Na Smectites and Swelling Chlorite. The presence of swelling clays in the samples could have an important influence on the stability of these soils in the field and this will be discussed further in Section 3.5.3. To fully assess the impact that swelling clays will have on the soil requires determination of the percentage of each clay mineral in the sample. This is beyond the scope of this study and was not attempted.

3.5.2.3 Natural moisture contents, Atterberg Limits and related indices.

Natural water contents, liquid and plastic limits, plasticity index and activity for the same 5 soil and 5 colluvium samples were determined in accordance with New Zealand Standard (NZS) 4402 (parts 2.1, 2.2, 2.3). Testing used the standard Casgrande cup and involved only the sand, silt and clay component. Particles larger than sand size were removed during sample remoulding. Although samples were immediately sealed in plastic bags following sampling, it is expected that some drying out of the soils occurred during storage.

Table 3.9 Summary of Atterberg Limits and related indices

Sample	Depth (m)	Type	Atterberg Limits LL	Pl	PI	Activity	Mc
Stillwater L3	0.35	c	92	42	50	1.388	68
Stillwater L4	0.35-0.57	c	95	60	35	1.207	72
Stillwater L5	0.57-1.0	c	81	55	26	1.08	74
Omoto L2	0.1-0.26	r	79	37	42	2.333	49
Omoto L3	0.26-0.72	r	73	35	38	1.118	50
Runanga L3	0.53-0.98	c	88	42	46	0.65	49
Runanga L4	0.98-1.43	c	77	40	37	0.95	43
Kaiata L3	0.26-0.43	r	74	44	30	1.2	55
Kaiata L4	0.43-0.83	r	73	41	32	1.29	54
Kaiata L5	0.83-1.23	r	48	31	17	0.68	35

Note: 1) "r" denotes residual soil and "c" denotes colluvial in origin.
2) Soil sample locations are given in the respective profiles.
3) Mc denotes natural soil moisture content.

Liquid limits (Table 3.9) for the samples were high, range 48-95 and plastic limits were also high, ranging from 31-60. Plasticity index values for the samples ranged from 17-50. Activity data for the soil samples are presented in Table 3.9. and range from inactive to active (after Skempton 1953).

Soil moisture values for both Stillwater and Omoto samples increased with depth, while moisture values for both Runanga and Kaiata Hill Soils decreased with depth (see Table 3.9). As indicated earlier, it was expected that some drying of the samples occurred during storage. It is therefore probable that the trend of decreasing moisture content resulted from the samples drying during storage as opposed to a natural trend. The natural moisture contents determined for Kaiata and Runanga Hill soils are regarded as anomalous.

3.5.3 STABILITY IMPLICATIONS.

The results of particle analysis and Atterberg Limits and related indices for the 5 colluvium and 5 regolith samples is shown in Table 3.10. Silt formed the dominant size fraction with clay sized particles comprising the remaining majority of the samples. Therefore these soils have been renamed as Clayey SILTS. The large clay content and the presence of swelling clays (notably Runanga Hill Soils) must contribute to the frequency with which the soils fail in the field under intense or prolonged rainfall. Swelling clays will absorb water which will increase the surcharge on the soil and decrease the soil cohesion that is resisting shear. Failure of the soil may occur when the shear strength of the soil is exceeded.

Atterberg Limits are dependent on the clay mineralogy of the sample although there is no single liquid limit (or plastic limit) that is characteristic of a particular clay mineral (Scott 1980). This results from the inherent variations of structure and composition within the clay mineral lattice and from the variation in exchangeable-cation composition (Bell 1983). There does not appear to be a correlation between the Atterberg limits and the clay mineralogy (Table 3.11). The activity of the soil is the ratio of the plasticity index to percentage clay sized particles (Beavis 1985). Bell (1983) indicates that there is only a general correlation between the clay mineral content and the activity of a soil. Kaolinitic and illitic clays are the least active whilst montmorillanitic clays range from inactive to active. Active clays have a relatively high moisture holding capacity and a low resistance to shear. Activity data obtained for the samples (Table 3.11) are highly variable and there does not appear to be a correlation between the clay mineralogy and the activity of the soil.

The liquid limits and plastic limits for the samples have been plotted on a Casgrande plasticity diagram (Figure 3.9). The samples plotted under the "A" line classifying them as silts of very high to extra high plasticity (using the USCS classification). The classification of the samples as Clayey SILTS by particle size analysis is in agreement with the classification obtained by the Casgrande diagram.

Soil moisture values (Table 3.10) increase with depth, although as indicated, the trend for Kaiata and Runanga samples is regarded as anomalous. Field observation of landslides involving regolith indicated that the bedrock/soil interface was an important discontinuity along which sliding takes place. The lower sample for each of the soils was obtained from immediately above bedrock and contains the highest soil moisture contents (or is assumed to). Therefore high soil pore pressures along this interface probably contribute to the destabilisation of these soils. As these soils are free draining (based on field observation), high pore pressures probably occur only during intense or prolonged rainfall.

Natural moisture values were close to the liquid limit and as sampling followed a period of fine weather, it is likely that natural moisture values exceed the liquid limit under heavy or prolonged rainfall, thereby contributing to a substantial decrease in the shear strength of the soil. This must contribute to the destabilisation of these soils in the field.

Table 3.11 Clay mineralogy data summary

Sample	LL	A	Clay mineralogy
Still L3	92	1.388	Fe Chlorite, Kaolinite, Sepiolite
Still L4	95	1.207	Kaolinite, Fe Chlorite, Sepiolite
Still L5	81	1.08	Kaolinite, Fe Chlorite, Sepiolite, Mica
Omo L1	79	2.333	Fe Chlorite, Kaolinite, Mica
Omo L2	73	1.118	Fe Chlorite, Kaolinite, (Swelling Chlorite or Na Smectite)
Run L3	88	0.65	Swelling Chlorite, Hollysite, Kaolinite, Mica
Run L4	77	0.95	Swelling Chlorite, Fe Chlorite, Mica, Kaolinite, Imogolite
Kai L3	74	1.2	Fe Chlorite, Kaolinite, Na Smectite
Kai L4	73	1.29	Fe Chlorite, Kaolinite, Na Smectite
Kai L5	48	0.68	Fe Chlorite, Kaolinite, Sepiolite, Mica

Note: LL = Liquid limit.
A = Activity of the soil.

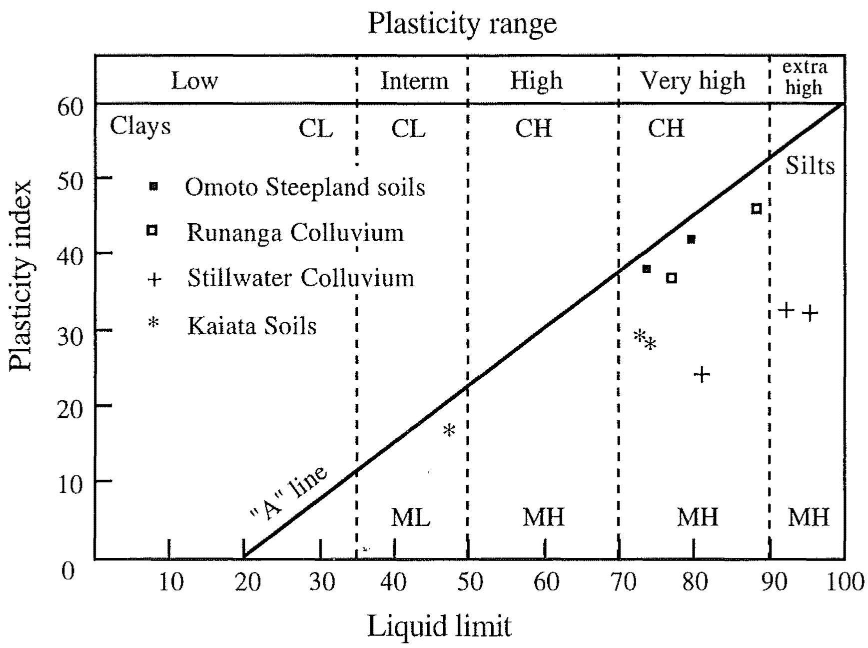


Figure 3.9 The distribution of plasticity values for regolith shown on a Casgrande plasticity diagram.

Table 3.10 Summary of the laboratory testing of the residual and coluvial samples

Sample	Depth (m)	Type	M_c	Particle size (%)			Atterberg limits				USCS
				Clay	Silt	Sand	LL	PL	PI	Activity	
Still L3	0.35	c	68	36	59	5	92	42	50	1.388	MH
Still L4	0.35-0.57	c	72	29	61	10	95	60	35	1.207	MH
Still L5	0.57-1.02	c	74	24	69	7	81	55	26	1.08	MH
Omo L2	0.1-0.26	r	49	18	72	18	79	37	42	2.333	MH
Omo L3	0.26-0.72	r	50	34	55	11	73	35	38	1.118	MH
Run L3	0.53-0.98	c	49	71	21	8	88	42	46	0.65	MH
Run L4	0.98-1.43	c	43	39	38	21	77	40	37	0.95	MH
Kai L3	0.26-0.43	r	55	25	60	15	74	44	30	1.2	MH
Kai L4	0.43-0.83	r	54	24	62	14	73	41	32	1.29	MH
Kai L5	0.83-1.23	r	35	25	65	10	48	31	17	0.68	ML

Note: 1) "r" denotes residual soil and "c" denotes colluvial in origin.
2) "M_c" denotes natural moisture content of the soil.
3) Sample locations are shown in the respective profiles.

3.6 SYNTHESIS.

Chapter 3 has detailed the engineering geological and geotechnical investigations that were carried out in order to assess landsliding within the field area. Field investigations were centred around mapping (at a scale of 1:10 000) geological features and landslides identified from the interpretation of aerial photographs. Aerial photographs taken at different scales and times over the last fifty years provided the primary source of information on the distribution of landslides within the mapped area. Field investigations involved sampling of rock mass features, field descriptions and sampling of rock and soil lithologies.

Point load strength and slake durability testing ascertained that lithotypes forming the Cobden Limestone had a high to very high resistance to erosion and moderate rock strength. From mechanical strength testing the limestone appears to be a suitable foundation material although there are a range of defects that could act to destabilise the rock. Stillwater Mudstone had low strength and very low resistance to erosion. Defect orientations within the rock proved impossible to measure although these are also expected to contribute to the instability of the rock. Stillwater Mudstone would not appear to be highly suitable as a foundation material.

Some soil samples, notably Runanga Soils contained significant concentrations of Swelling Chlorite and Na Smectites were detected in other soils. These are expected to contribute to the frequency with which these soils fail. Natural moisture contents increased with soil depth and were close to the liquid limit of the samples. It was concluded that soil moisture probably exceeds the liquid limit during heavy or prolonged rainfall thereby decreasing the shear strength of the soil.

CHAPTER 4.
LANDSLIDE FAILURE MODELS.

4.1 INTRODUCTION.

Landslide types that have been identified in the Greymouth area may be grouped under the Varnes (1978) classification as: translational rock block slides and rock slides; translational debris or earth slides; rotational debris or earth slumps; flows; falls and rock avalanches. This chapter examines the causes of slope instability in the Greymouth area as inferred from general field investigations and from two site specific case studies, these being the Stanton Crescent and Australasian Hotel Slides identified by the author. Failure models for these two landslide types are then developed, and in addition, a review of the investigation programme that has been conducted on the Omoto Slip is presented with some comments as to the likely extent of that failure. This chapter concludes with a discussion on the implications that the identified failure types present for residential development.

4.2 LANDSLIDE TYPES.

4.2.1 BEDROCK FAILURES.

Landslides in the Greymouth area occurring in bedrock are typically defect controlled. In terms of the Varnes (1978) classification (see Section 1.5.2), these are translational failures and may be grouped into rock block slides, rock slides and rock avalanches. These failures involve sliding along bedding with releasing along joints in the head and lateral areas of the slide.

1. Rock block slide. Only one rock block slide was identified in the field area (grid reference 633590 Figure 3.3c, map pocket at back). The block (of Puketahi Mudstone member) is approximately 270m in length, 80m in width and some 20-30m in depth and appears to have slid (essentially intact) some 20-30m down slope along bedding. No field evidence of lithological control or bedding plane shear was observed in the field. However, the strike and dip (170/25° WSW) of the limestone in this locality is consistent with sliding parallel to bedding. The lateral releasing margin is thought to be a planar and persistent joint trending 050° and dipping steeply to the south-east. The landslide is clearly defined on the 1943 aerial photographic set but is obscured in the field by the grow-back of vegetation. Given the size of the slide, the most likely trigger is a very large earthquake.

2. Rock slides. Translational rock slides such as the Stanton Crescent Slide (detailed in Section 4.4) range from 100 - >350m in width, and 90 - 350m in length (parallel to slope). The depth of identified rock slides could not be determined but is anticipated to be over 20m in some cases. Depth determination would have required cored drilling which was beyond the financial resources of this study. Rock slides are typified by a poorly - well developed crescent shaped back scarp, the development of a back graben, well defined lateral margins and an absence of rotation within the slide mass. The back graben may extend from <5m to over 20m in depth. Field investigations suggest that the failure surface is bedding although a lack of suitable exposure meant that this could not be confirmed.

3. Rock avalanche. A large rock avalanche that has occurred within the Tarapuhi Limestone Member was identified from aerial photographic interpretation (grid reference 635590 Figure 3.3c in map pocket at the back). The source area for this slide is immediately to the south of Trig point FB and the slide debris extends (parallel to slope) 600-700m in a south-west direction down the slope. The slide debris (Figure 4.1) has formed an area of blocky limestone colluvium that lies in contact with a block slide (discussed above). The failure surface of the avalanche appears to be parallel or subparallel to bedding indicating some lithological or bedding plane shear control on the development of the slide. However, this could not be confirmed in the field owing to the dense vegetation that covers the slope. The avalanche is clearly visible on the 1943 aerial photographic set but is obscured on the latter aerial photographs by the grow-back of vegetation. The most likely trigger for the slide was an earthquake and from the morphology of the avalanche, it is likely to be pre-European in age.

4.2.2 SURFICIAL FAILURES.

1. Slides. Landslides in the Greymouth area involving surficial material may be grouped (based on Varnes 1978) into slides (both translational and rotational) and flows. Translational failures (such as the Australasian Hotel Slide) may be classified as either shallow (<2m) or deep (>2m) dependent on the thickness of the surficial material and are included in the debris or earth slide category of Varnes (1978), dependent on the materials involved. The failure surface typically forms the bedrock/surficial material interface. In areas of colluvium, the failure surface is well defined and involves colluvium sliding over essentially unweathered parent material. The failure surface of the slide is generally greater than 2m in depth. In areas of *in situ* regolith, the failure surface is generally less well defined and is less than 2m in depth. Translational debris slides range from 20-150m in length (parallel to slope) and 4-50m in width at the toe. Translational failures are characterised by well defined lateral margins, concave back scarps and the absence of any rotation of the slide mass.

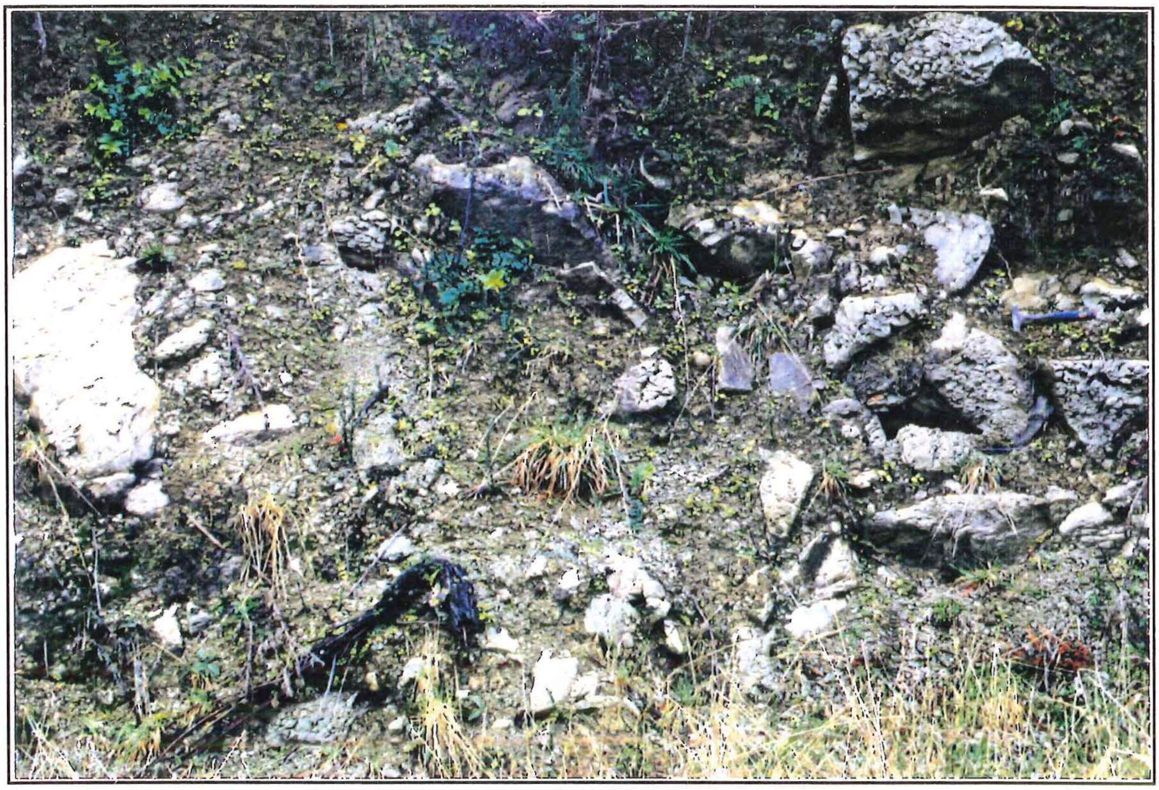


Figure 4.1 Rock avalanche debris forming a blocky limestone colluvial deposit.

Rotational slides in the Greymouth area are included in the debris or earth slump category of Varnes (1978). These slides are generally areally discrete (around 10m in width and length), the slide surface is generally less than 2m in depth and are often confined to the surficial material. In areas of shallow surficial material (<2m depth), the slide plane may involve the bedrock/surficial material interface. The landslides are characterised by a concave backscarp and a rotated block of slide debris. Rotational failures often incorporate a component of flowage as the failed mass undergoes self drainage near the landslide toe.

The most recent activity to have taken place on the Omoto Slip is included in the rotational failure category. However, the failure is deeper seated and more areally extensive than the small rotational slumps that are observed in the field. Further details on this slide are provided in Section 4.3.

<u>2. Flows.</u> Debris flows (after Varnes 1978) are the most frequently occurring failure type identified in the field area and these occur on even the most heavily vegetated slopes, where slopes exceed 15°. This type of failure is characterised by the inferred mechansim of movement (as a flow) and commonly has a spoon shaped source area and a well defined channel. These flow failures are often initiated as a translational slide but are thought to behave as a flow once in motion. The length of these failures is variable from <2m to over 60m in length and the width is generally less than 10-15m. They are shallow seated failures generally occurring in surficial material <2m in depth. The landslide debris commonly spreads out over a wide area at the toe of the slide.

4.2.3 FALLS.

Rock falls (following Varnes 1978) involving Cobden Limestone are relatively common around the Grey River Gorge and on other steep and exposed limestone slopes. The smaller <0.1m^3 failures occur on a regular basis while the larger, 15m (height) x 5 (width) x 1m (depth) slab failures occur less frequently (Figure 4.2). The development of these failures is a function of the defects present within the limestone. Failures commonly incorporate an element of topple or basal sliding along steeply inclined defects, with lateral release along vertical or subvertical joints. Falls generally involve a combination of the major defects identified as part of rock mass characterisation (Section 3.4). Iron stained joint walls indicate that ground water is present in the joint walls, therefore high pore pressures can be expected to have an important influence on the development of these failures.

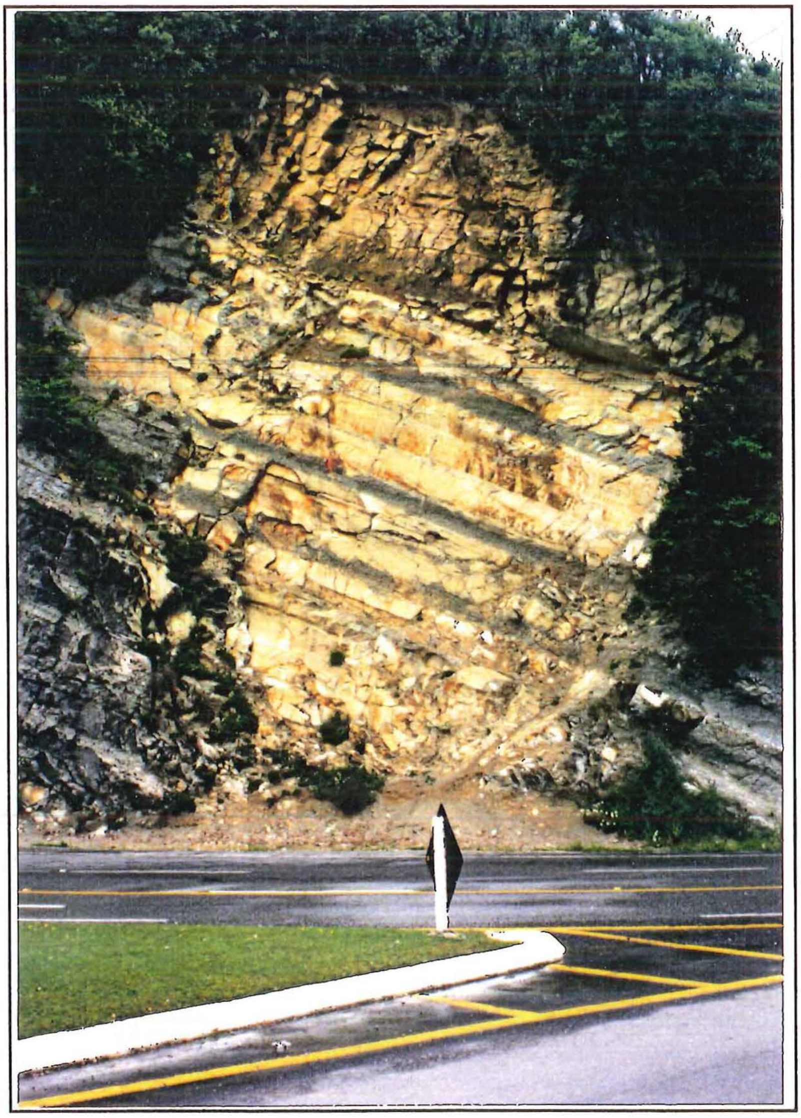


Figure 4.2 A large rock fall involving Cobden Limestone that has occurred at the Greymouth side of the Cobden Bridge. Note jointing within the limestone and iron stained joint faces. The prominent joint trending parallel to the orientation of the photograph is the F1 joint identified during rock mass characterisation.

4.3 OMOTO SLIP - A REVIEW.

4.3.1 BACKGROUND.

The Omoto Slip is the largest landslide present in the study area. Although it was not investigated by this author, a review of previous engineering geological investigations and of the failure mechanism, is provided here.

Documented activity on the Omoto Slip (see Figure 2.1 grid reference 755876) dates from 1896 to the present time. However, the author has sighted newspaper reports (Grey River Argus) that indicate disruption of the road and rail links may have occurred as early as 1874. Continued disruption during the 1980's to State Highway 7 and the railway line (which cross the slip) led to a detailed investigation by the Department Of Scientific and Industrial Research (Paterson 1989) for the Ministry of Works and Development.

4.3.2 INVESTIGATIONS.

Surface mapping by McLean (1987) was restricted to McKendrys Corner above and below the highway (Figure 4.3), in particular to around existing housing developments on the slide. A programme of 5 cored drill holes and 3 auger holes to install piezometers was proposed. This subsurface investigation programme was based on the assumption that the slip was a shallow-seated failure and did not extend above the highway. A record of ground water levels within the slide indicated that water levels rise in response to precipitation and to flooding within the Grey River. In addition it was discovered that a perched water table was present close to the ground surface following heavy rain. Subsurface samples collected during drilling were tested for grain size, ring shear and standard penetration tests were also carried out. Two samples of colluvial material (from the assumed failure surface) contained 60-70% silt plus clay and had residual friction angles of 19.1° and 20.6° and apparent residual cohesion values of 12.9 kPa and 13 kPa respectively (Paterson 1989).

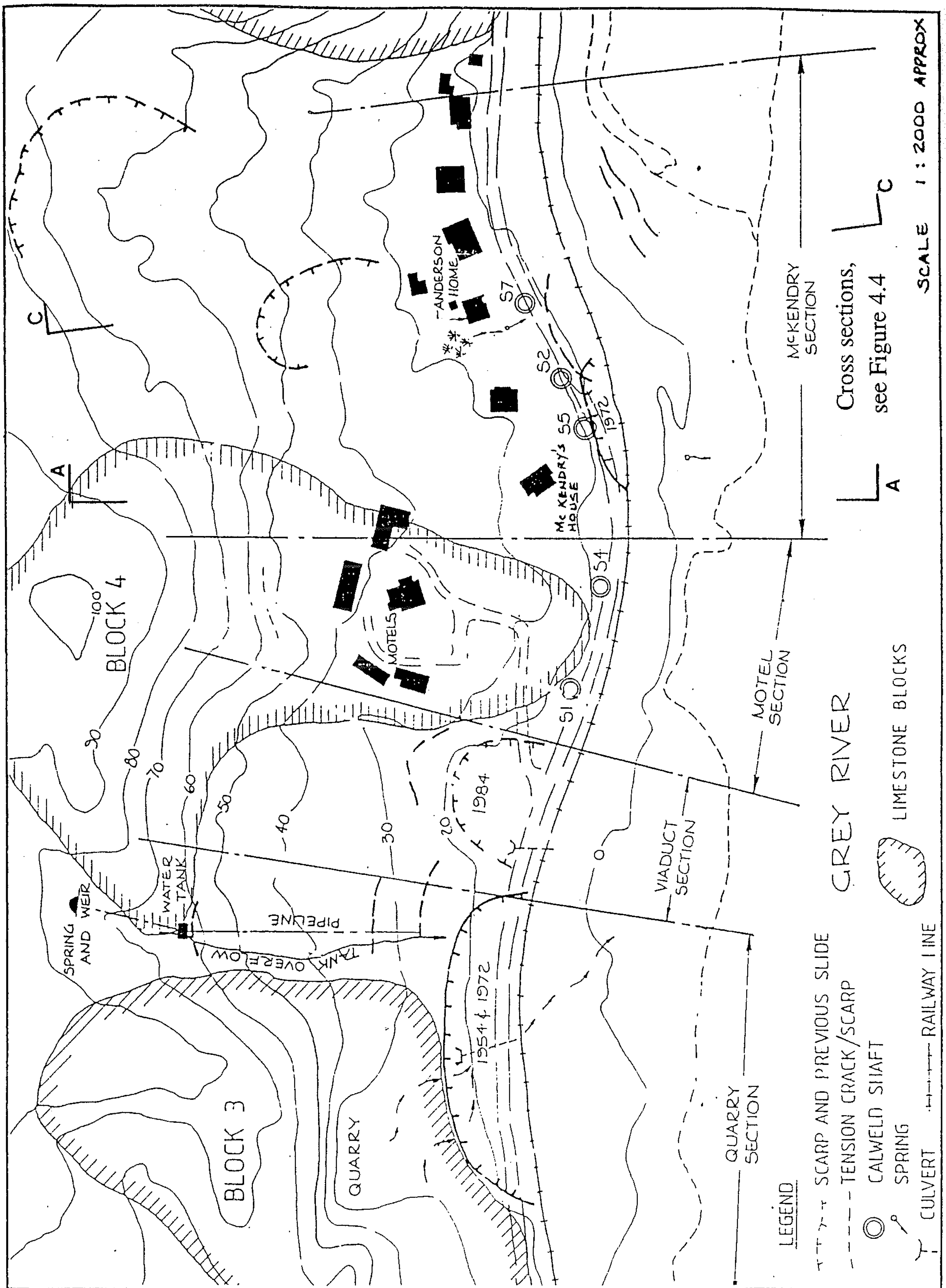
4.3.3 GEOLOGICAL INTERPRETATION.

Interpretation of the subsurface geology of McKendrys corner from the surface down is as follows:

1) colluvium, 4.5-15.5m thick, consisting of mainly mudstone and minor limestone fragments in a silty clay matrix;

2) limestone scree deposit, 7.5-15m thick consisting of angular limestone fragments in a silty clay sand matrix;

3) alluvium, 3.5 m+ consisting of unweathered greywacke and igneous clasts; and



Title
Location of McKendry's corner.

WORKS Consultancy Services	Location: SH 7 OMOTO SLIP, GREYMOUTH	Job No.
	Client: Transit New Zealand	Figure 4.3

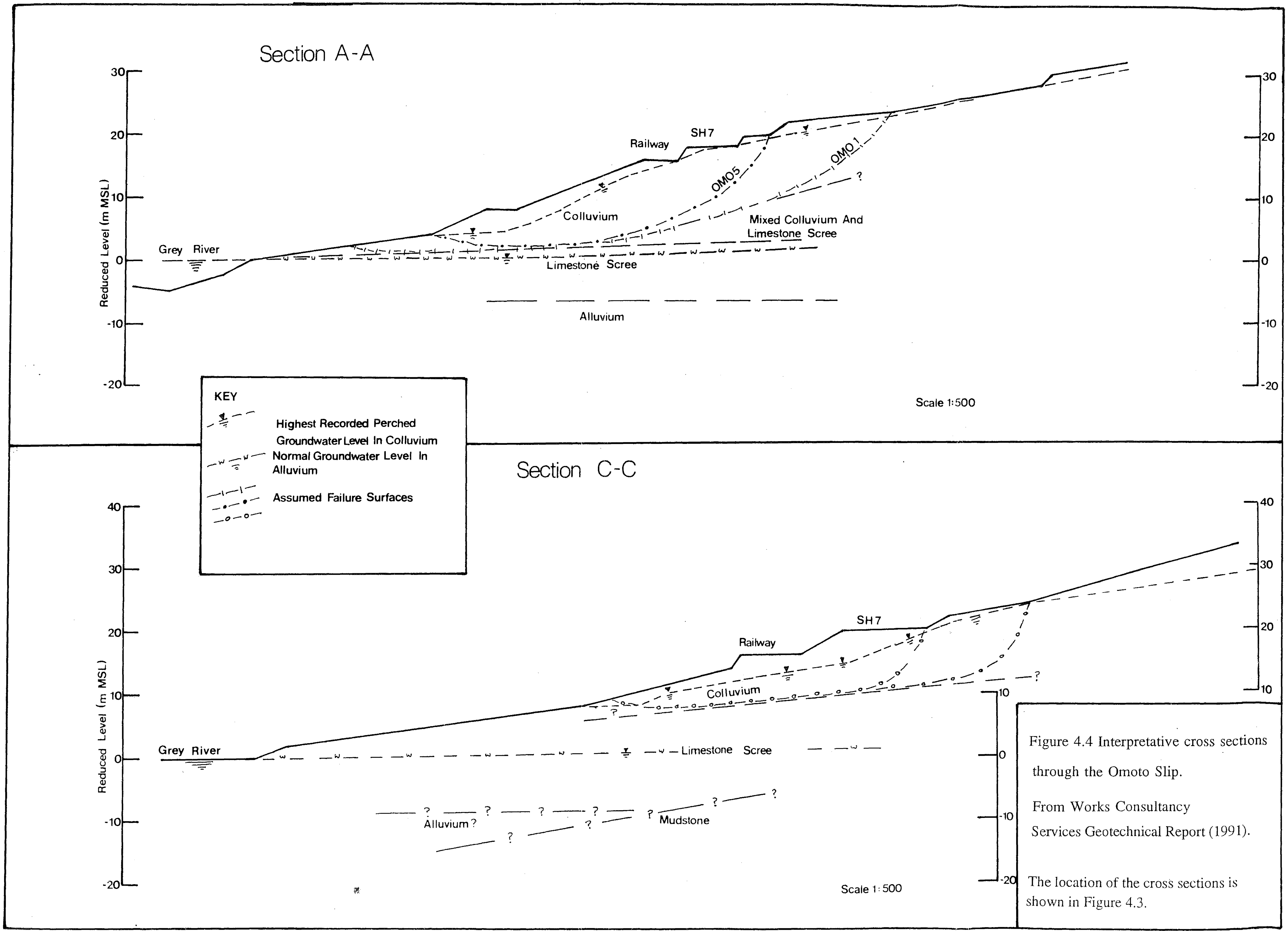


Figure 4.4 Interpretative cross sections through the Omoto Slip.

From Works Consultancy Services Geotechnical Report (1991).

The location of the cross sections is shown in Figure 4.3.

4) mudstone, (Kaiata Formation) 2.7m+ consisting of unweathered grey-brown calcareous mudstone (Paterson 1989).

Kaiata Mudstone constitutes the bedrock in the area and has been eroded by the Grey River in recent times (Paterson 1989). River gravel was deposited on the eroded mudstone surface and limestone scree accumulated at the foot of the escarpment as the river course migrated north. During this time it is likely that large blocks of limestone became separated from the escarpment and slid downslope. Subsequent weathering of mudstone exposed below the escarpment has formed an unstable colluvium which has moved downslope covering the limestone scree at the base of the slope (Paterson 1989).

4.3.4 SLOPE STABILITY ASSESSMENT.

The investigation by Mclean (1987) indicated that active slope movement is confined to the superficial layer of colluvium as shown in Figure 4.4 by Paterson (1989). There was no evidence of instability within the formations that underlie the colluvium (Mclean 1987). In most of the drill holes there is a zone of plastic clay present between the colluvium and the limestone scree and this is assumed to be the failure surface within the landslide complex (Paterson 1989). The failure mechanism of the landslide is semi-rotational in nature (Figure 4.4).

4.3.5 COMMENT.

The investigation that has been outlined above was concerned with the most recent landslide activity in the area and determined that this is confined to the colluvial material. However, it is highly probable that the Omoto Slip is a much deeper seated wedge failure and that the failure plane(s) are located within the Kaiata Mudstone. The possibility exists that this deeper seated failure has been triggered by an earthquake. A comprehensive investigation in addition to those already carried out would be required to determine whether deep seated failure is, or has, occurred.

4.4 STANTON CRESCENT SLIDE.

4.4.1 GENERAL.

The Stanton Crescent Slide is located (Figure 4.5) between Rata Street and Power Road on State Highway 6 and incorporates the residential subdivision of Stanton Crescent. It is a deep seated, translational rock slide failing along bedding in Stillwater Mudstone. The landslide was initially identified by aerial photographic interpretation during the 1:10 000 scale mapping of the field area and was chosen over other similar failures for detailed study because of the relative site accessibility. The objectives of the investigations were to determine mode of failure for this landslide type, rather than assess it's present activity.

Investigation of this slide involved a detailed examination of aerial photographs (see Table 3.1), engineering geological mapping (at a scale of 1:1500) and limited subsurface investigation by auger hole profiling to determine the failure surface.

4.4.2 SITE DESCRIPTION.

Bedrock consists of Stillwater Mudstone (bedding attitude approximately 173/20° WSW) and is overlain by slope colluvium. The terrace surface above consists of *in situ* Karoro Formation and the sands and gravels on the seaward side of the Greymouth-Ross railway form the older marine phase of the Nine Mile Formation (see Figure 2.1).

Much of the mapped area has been covered in residential development and farm land is present to the south of the area (Figure 4.5). Areas of the slide not currently developed (for housing), or too steep to be developed, are covered in regenerating scrub. State Highway 6 and the Greymouth-Ross railway cross the toe of the slide (Figure 4.5). A large displaced block of Stillwater Mudstone is present to the north of the mapped area and this is inferred by the author to be a relict of a much older, but similar type of landslide to the one under study.

4.4.3 ENGINEERING GEOLOGICAL MAPPING.

The slide is equidimensional, approximately 350m in length and width (at the toe) (Figure 4.6). A very large crescent shaped back scarp 60m in length and 20-30m in height dominates the head area of the slide (Figure 4.6). Two streams, originating from the apex of what was once a well-defined graben, flow down each of the lateral margins. Aerial photographic interpretation showed that large (20x20x5m) detached but intact blocks of slide debris were present in the graben area. These have now been flattened by earth works associated with residential development and much of the graben area has been infilled.

The toe of the slide lies on the western side of State Highway 6 and from aerial photographic interpretation it would appear that the toe is located within or is overlain by the marine deposits of the older part of the Nine Mile Formation (Figure 4.6). Field investigation supports this hypothesis, although much of the area has been extensively modified by urban development. However, the exact extent of the slide toe could not be determined in the field.

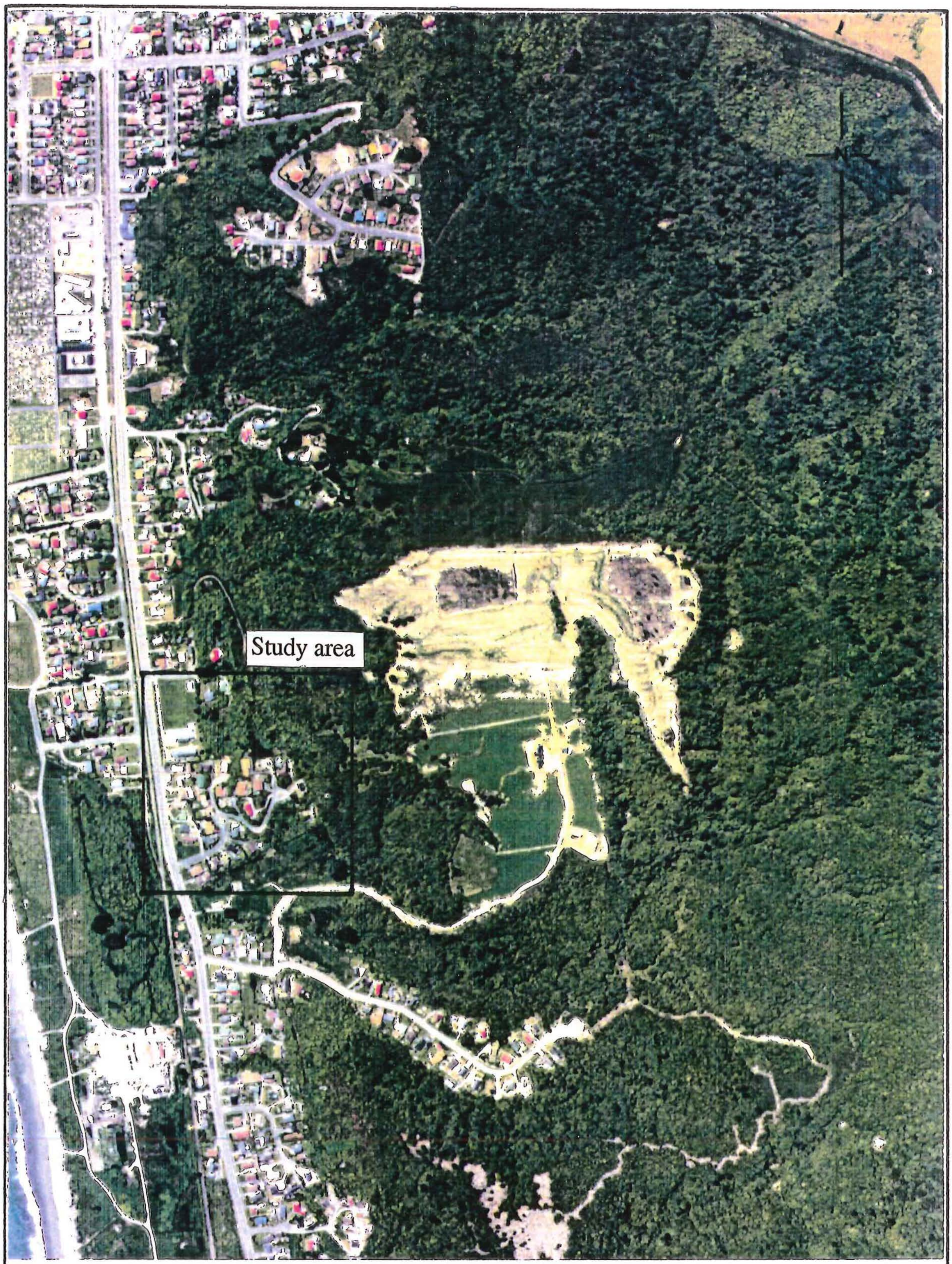


Figure 4.5 A vertical aerial photograph showing the location of the Stanton Crescent Slide field area. The photograph was taken in 1992 by the West Coast Regional Council at a scale of 1:10000.

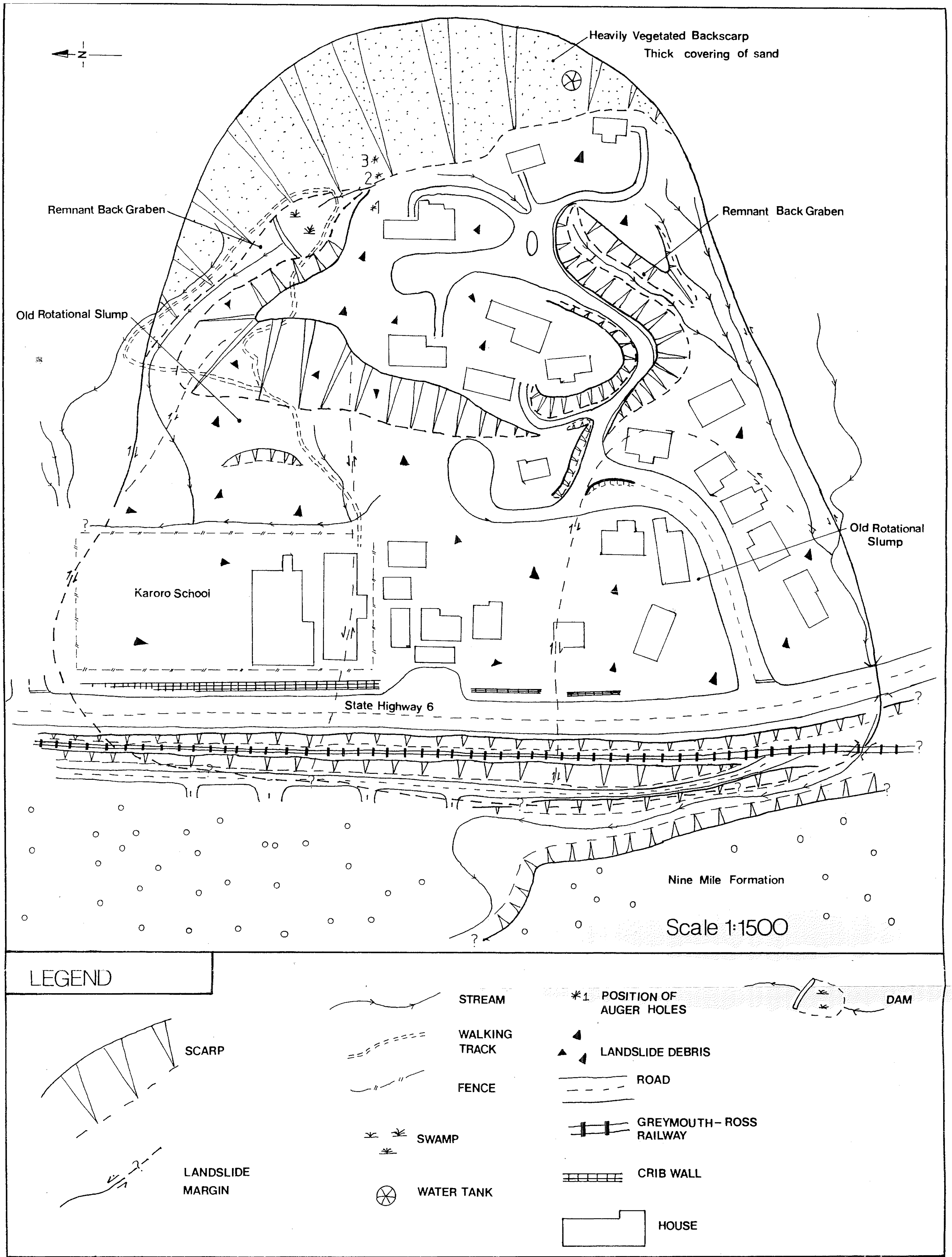


FIGURE 4.6 ENGINEERING GEOLOGICAL MAP OF STANTON CRESCENT.

Landslide debris consists of moderately weathered, displaced Stillwater Mudstone (observed in several roading batters and in the lateral slide margins), overlain by unconsolidated sand and regolith formed *in situ.*

Aerial photographic evidence suggests that two rotational-flow type failures (Figure 4.6) have occurred in the lower half of the slide. Little evidence remains of the failure to the south, but some morphological features remain of the failure in the north, including a remnant head scarp and a highly degraded lateral margin (Figure 4.6).

Storm water overflow pipes from housing and roading sources discharge into the head scarp and lateral margins of the slide, although there is no evidence for current movement.

4.4.4 SUBSURFACE INVESTIGATION.

It was originally hoped to be able to exhume the failure surface of the slide in the head scarp region by backhoe excavation and three exploratory auger holes were put down to assess the viability of this option (Figure 4.6). The profiles (see Appendix A3.1) revealed that more than 3m of unconsolidated sand (eroded from the Karoro Formation on the terrace surface above) is present overlying the head scarp. It was felt that with bedrock at an unknown depth (but >3m) that backhoe excavation was unlikely to be successful. Site access was also a problem given the existing residential development and with the disturbance of the area that has accompanied development, it was felt that the site was unsuitable.

4.4.5 AGE AND PRESENT ACTIVITY.

The age of the landslide is difficult to establish with any certainty, but it probably relates to the formation of the coastal escarpment. As outlined previously, the escarpment was formed by marine processes (*c.* 4000-5000 years B.P.) during the Holocene and was subsequently uplifted to it's present position. Since the toe of the slide is assumed to be within or beneath Nine Mile Formation sediments (*c.* 4000-5000 years B.P.), the slide must have formed prior to the uplift of this the older phase of the Nine Mile Formation, i.e. before 4720 $\pm$ 70 years (Suggate 1968). Undercutting of the slope toe by marine processes, was presumably responsible for allowing the slide mass to slide along bedding (assumed). However, the landslide could also have been generated by an earthquake, or a combination of these factors.

There was no field evidence to support recent movement of the landslide. Sediments forming the Nine Mile Formation appear to be acting as a toe buttress, preventing further movements of the landslide downslope. However, the effects of storm water from residential and roading sources on the stability of the landslide cannot be ascertained. There is a possibility that added storm water associated with future residential development could cause further movement.

4.4.6 FURTHER WORK REQUIREMENTS.

Supplementary work is required to accurately locate the failure surface and confirm the mode of failure, the need for cored drilling is noted. However, given that the failure layer may be very thin (for example <0.2m in thickness), and due to the fractured nature of the Stillwater Mudstone then even drilling may not be successful in locating the failure surface.

There is a very clear need to determine the affects of existing storm water on the stability of the landslide and of the effects of added residential development prior to additional subdivision. Control of the existing storm water system is recommended in order to maintain stability of the landslide.

Fixed monitoring points should be emplaced at suitable locations on the Stanton Crescent Slide and surveyed on a regular basis (every six months for example). These would detect any future movement of the slide mass and possibly provide some warning as to any future large scale movements.

4.5 AUSTRALASIAN HOTEL SLIDE.

4.5.1 GENERAL.

The Australasian Hotel Slide is located behind the Australasian Hotel on Paroa Road (Figure 4.7) and consists of a deep seated (>2m) translational debris slide. The landslide was initially identified from aerial photographic interpretation as part of the general (1:10 000) mapping, and was chosen over other similar failures for detailed study because of site accessibility and the proximity to an elevated area from which a survey base station could be established. The objectives of the investigation were to:

1) conduct a detailed engineering geological investigation of the slide determining the mechanism of failure;

2) sample and geotechnical characterisation of materials involved in failure; and

3) develop a failure model and extend this to other similar failures in the study area.

Based on the these three objectives, the investigation programme included field mapping (at a scale of 1:1000), monitoring of survey markers, subsurface investigation (auger holes and seismic refraction) and laboratory testing of materials recovered.

Figure 4.7 A vertical aerial photograph illustrating the location of the Australasian Hotel Slide field area. The photograph was taken in 1992 at a scale of 1:10000 by The West Coast Regional Council.

4.5.2 SITE DESCRIPTION.

Slide debris lying on a west facing coastal escarpment extends from the crest to the toe of the slope. Bedrock is Stillwater Mudstone and the overlying colluvial material includes a complex of Stillwater and Runanga Hill Soils and material eroded from the Karoro Formation. Marine sands and gravels of the older phase of the Nine Mile Formation form the low terrace at the base of the scarp, while sediments of the Karoro Formation cap the terrace surface above.

Grass and sparse gorse cover the lower half of the slope, while a dense mixture of native and introduced scrub covers the upper half of the slope. Typical canopy cover is at about 3-5m in height and consists of tree ferns and regenerating scrub.

Interpretation of aerial photographs revealed that prior to 1943 the slope was cleared and fenced for grazing purposes. At present, only the lower half of the slope is used for agriculture. The land is currently zoned as residential in the Greymouth District Plan and urban development has occurred along both the toe (Paroa Road) and crest (Arnotts Heights) of the slope (Figure 4.7). No development has yet occurred on the slide area.

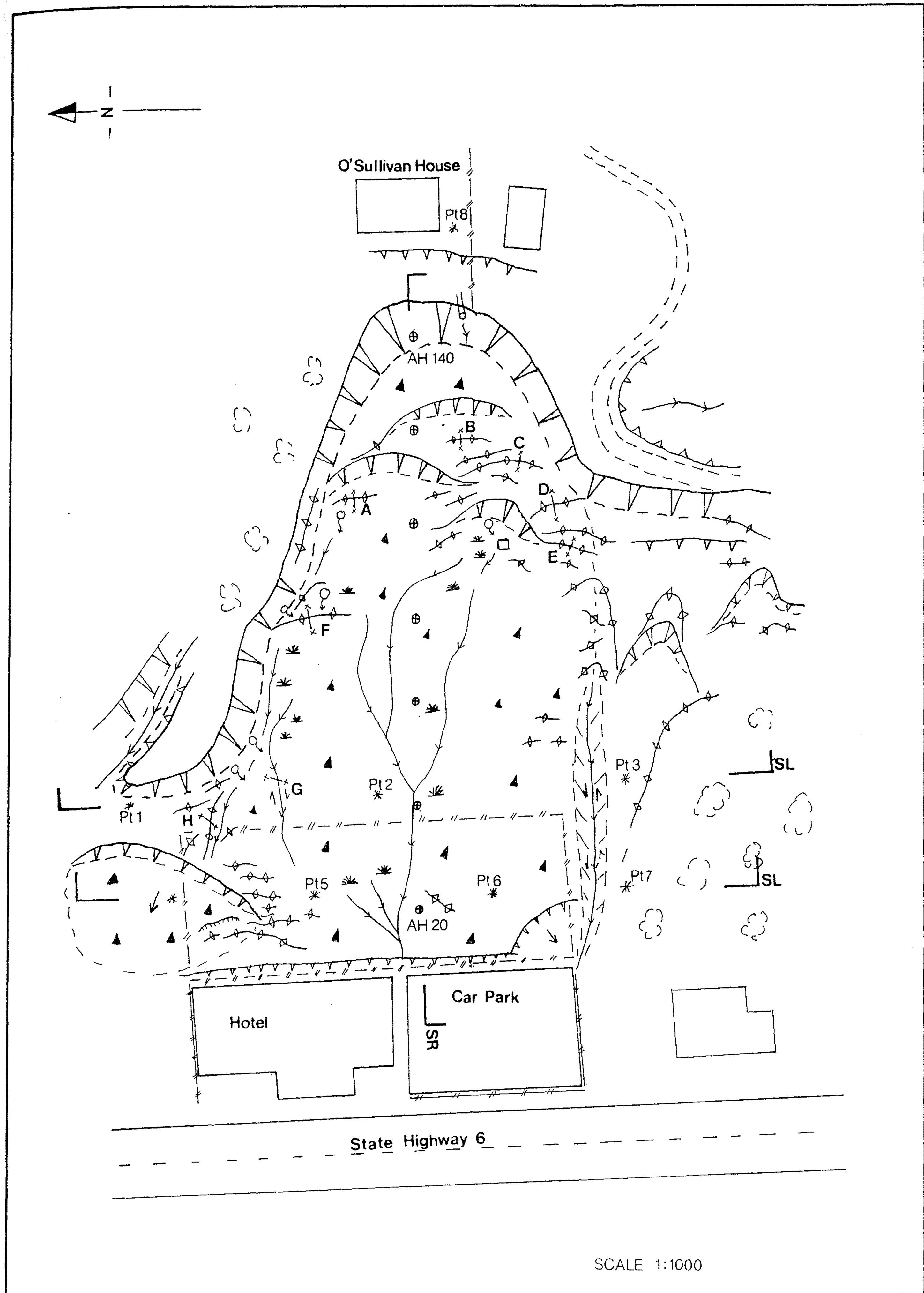
4.5.3 ENGINEERING GEOLOGICAL MAPPING.

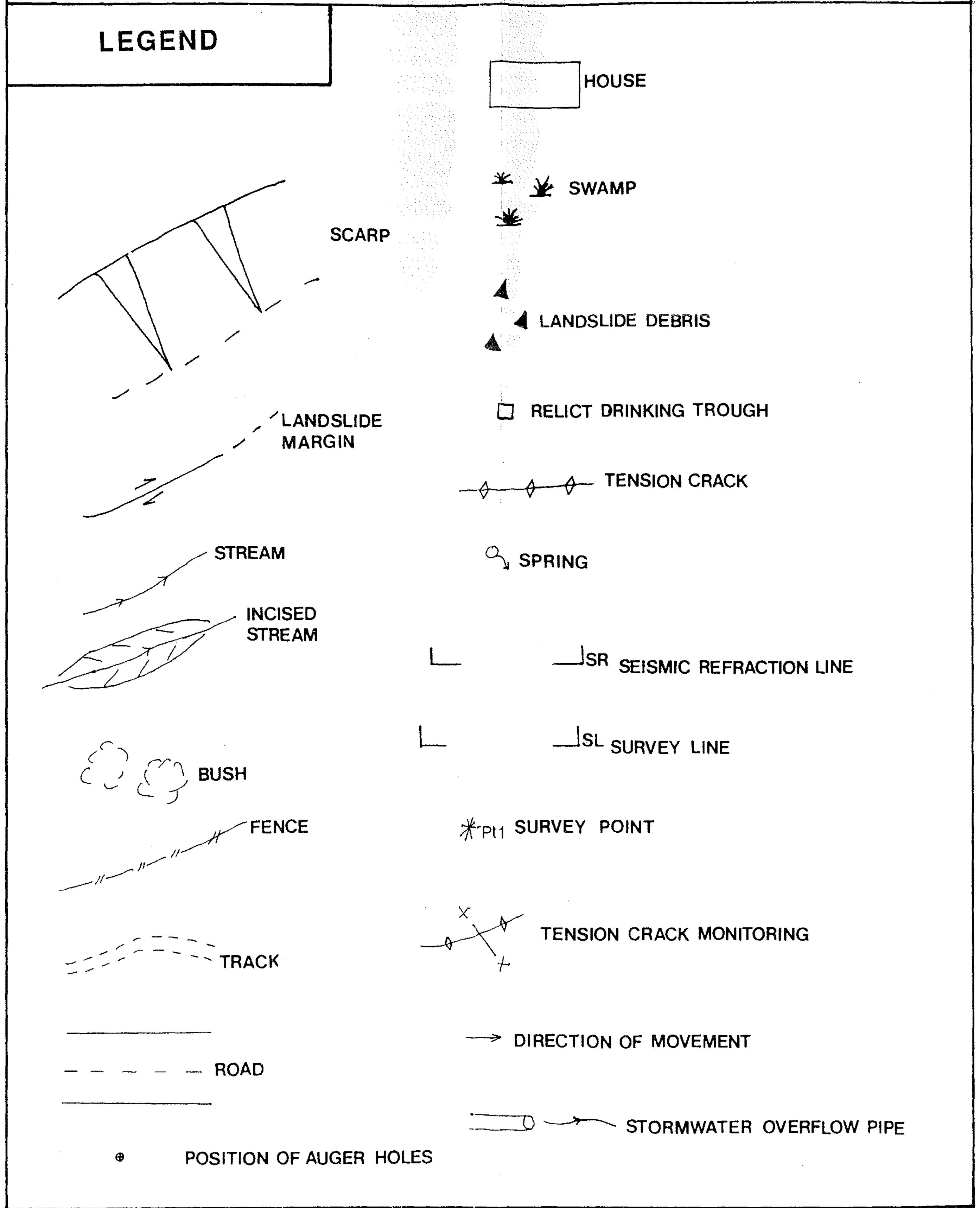
The landslide is 140m in length and at the toe of the slope is 60m in width (Figure 4.8). A prominent crescent-shaped head scarp 2-3m in height grades into a lateral scarp (1-2m in height) on the northern slide margin. The southern edge of the slide is marked by a small stream that has incised to bedrock. The source of the stream is a complex area of small rotational slumps and open (<0.3m deep) crescent shaped tension cracks of up to 20m in length (Figure 4.8). Flow in the stream occurs immediately following rain but otherwise the channel remains dry.

A small spring issuing from the middle of the head scarp supplies a constant source of water to a second stream traversing the middle length of the landslide (Figure 4.8). In the lower parts of the landslide, the stream has been channelled by drainage control measures. The stream flows "sheet like" over the middle and upper parts of the landslide and has created extensive areas of saturated ground. A similar situation occurs along the northern lateral scarp, where a second low volume spring has also created broad areas of saturated material (Figure 4.8).

The toe of the slope has been removed during the construction of the Australasian Hotel and adjacent car-park and a small 1-2m high, slightly degraded, batter is present (Figure 4.8). The slide base is assumed to be below the level of the car-park.

FIGURE 4.8 ENGINEERING GEOLOGICAL MAP OF THE AUSTRALASIAN HOTEL SLIDE





A small scarp is present in front of the O'Sullivan house at the crest of the slide. It is unclear whether this is due to regression of the slide, or whether it is due to the placement of fill during house construction. The latter is indicated by a small retaining wall formed from decaying tree ferns and waratahs. A storm water overflow pipe from the O'Sullivan house discharges into the headscarp of the landslide (Figure 4.8).

4.5.4 SUBSURFACE INVESTIGATION.

1. Auger holes and sampling.

A series of seven auger holes spaced at 20m intervals along the length of the landslide were used to obtain information on the nature of the slide debris and on the failure surface of the landslide. In addition the auger holes provided depth control for a seismic refraction profile that was carried out. The holes were logged from cuttings and some disturbed samples were collected and bagged for laboratory study. Full logs of these auger holes are given in Appendix A3.2.

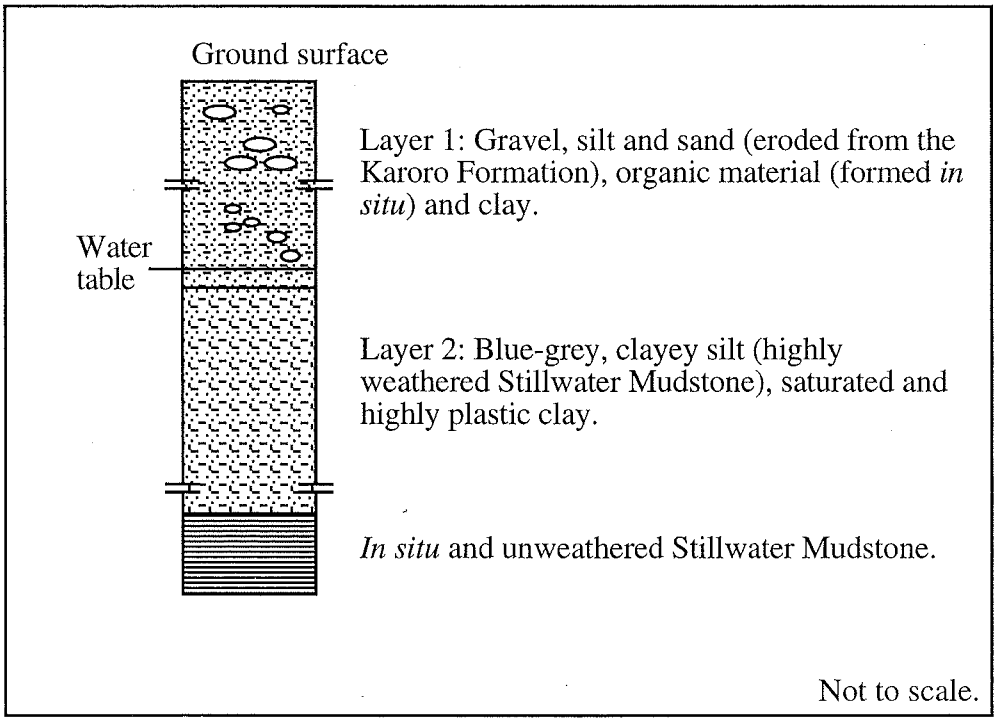


Figure 4.9 Generalised stratigraphy of the slide debris.

Auger hole profiling revealed that there was a consistent two layer stratigraphy to the slide debris (Figure 4.9) and that the water table was generally in Layer 1. The auger holes, although penetrating to bedrock in only two localities (auger hole AH120m and AH140m), failed to detect the presence of a discrete failure zone along which shear was obviously taking place. There are two reasons for this: 1) the nature by which the the hand auger hole samples the debris makes identification of thin horizons difficult; and 2) in most cases the auger did not penetrate for enough (maximum depth obtained 2.1m in AH20m, see Appendix A3.2). The absence of layer 2 in AH140m (see Appendix A3.2) suggested that sliding was taking place between layer 2 and bedrock. Therefore samples of layer 2 from AH40m and AH80m were collected for laboratory analysis.

2. Seismic refraction.

A seismic refraction profile was carried out on the surface of the Australasian Hotel Slide in an attempt to determine the depth to bedrock. The data obtained was then analysed by the reciprocal method of Hawkins (1961), the theory, methodology and results of which are given in Appendix A3.3.

Seismic refraction indicates that the depth to bedrock on the slide surface was in the range of 1.5-2.0m and that the bedrock surface roughly paralleled the ground surface. The depth to bedrock determined in auger holes AH100, 120 and 140 correlate with the refractor profile, but the remaining auger holes (AH20m, 40, 60, and 80) indicated that the refractor was within Layer 2. It would appear that the seismic refraction profile provides only a general indication of the subsurface geology of the landslide. Some lack of success of the profile is attributed to the high volume of traffic which travelled along the Greymouth-Ross railway and State Highway 6 on the day in which the profile was carried out. This contributed to the background noise and made detection of the first seismic signal difficult.

A second reason for the lack of success with the seismic refraction is that the subsurface of the slide may consist of material in which seismic velocity gradually increases with depth. If this is the case, then seismic refraction is unlikely to have provided sensible results. Further work (cored drilling) would be required to fully investigate the nature of the subsurface of the slide.

4.5.5 SURFACE MONITORING.

Surface movement instrumentation on the slide took a two fold approach: firstly, monitoring for surface displacement (Figure 4.10); and secondly, monitoring of tension cracks and suspected lateral shears. Installation of the survey network occurred during September/October 1992 and followed the completion of field mapping. Monitoring of the network continued until May 1993. The positioning of fixed monitoring points was based on the morphology of the landslide and was constrained by factors such as vegetation density and topography of the hill slope. Surveys were spaced at 1 month intervals, except for the final survey which succeeded the previous one by 3 months. The positioning of all monitoring points is shown in Figure 4.8.

The methodology is given in Appendix A3.4 and A3.5 and the results of the tension crack monitoring are shown in Table 4.1 and the survey network in Figure 4.11.

Figure 4.10 Surveying in progress on the Australasian Hotel Slide. The slide surface is outlined and the position of the prism is arrowed.

Movement of tension cracks (Table 4.1) is considered within the survey error and is not considered significant. Minor variations (in order of 1 or 2mm) in the distances between pegs was noticed in successive surveys. These are thought to represent ground swelling and shrinkage associated with wet and dry weather, rather than any displacement of the slide mass.

The displacement of fixed survey markers (Figure 4.11), in general, plot around a central point. These discrepancies are thought to represent survey error associated with movement of the prism during distance measurements. Small movements (1-5mm) of the pegs associated with landsliding would not have been detected by the method of surveying used. The recorded movement (Figure 4.11) appears random in orientation and is not thought to represent movement of the landslide mass. It is concluded that surveying of fixed pegs using a staff pole and prism is not accurate enough to allow detection of small landslide movements.

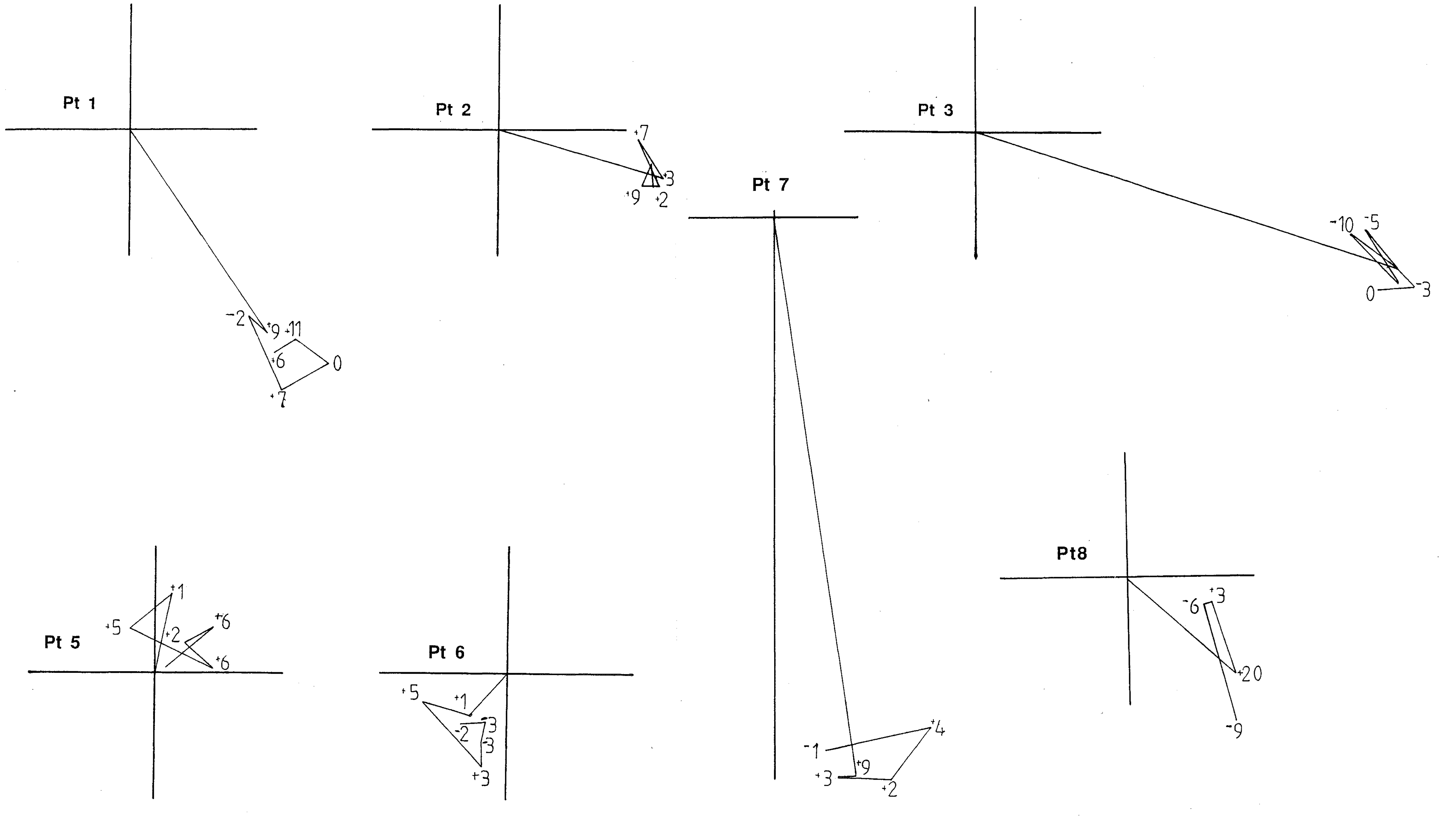


FIGURE 4.11 DIAGRAMATIC REPRESENTATION OF THE :JRVEY RESULTS

NOTE: Lines represent horizontal displacement vectors (mm) from false origin (which was based on initial survey). Numbers represent vertical displacement (mm), negative numbers = downwards.

Table 4.1 Results of the tension crack monitoring for the Australasian Hotel Slide.

	DATE					
Station	19/10/92	14/11/92	12/12/92	16/1/93	18/2/93	24/5/93
A	0.661m	0.657m	0.656m	0.656m	0.656m	0.655m
B	1.073m	1.07m	1.07m	1.071m	1.070m	1.07m
C	0.97m	0.972m	0.971m	0.972m	0.97m	0.971m
D	0.815m	0.814m	0.814m	0.815m	0.815m	0.814m
E	1.258m	1.257m	1.256m	1.256m	1.255m	1.256m
F	1.096m	1.095m	1.095m	1.096m	1.097m	1.96m
G	1.427m	1.442m	1.422m	1.0421m	1.422m	1.42m
H	1.328m	1.328m	1.329m	1.329m	1.329m	1.329m

4.5.6 GEOTECHNICAL CHARACTERISATION.

Laboratory testing of failure materials obtained during auger hole profiling was carried out concurrently with the soil characterisation phase of the study. The objective of the testing was to geotechnically characterise the failure materials. The following tests were carried out:

1) particle size analysis according to Lewis (1984);
2) Atterberg limits and related indices according to NZS 4402 (4.1, 4.2, 4.3); and
3) clay mineralogy.

The clay mineralogy graphs, the interpretation of these and the results of the particle size analysis are given in Appendix A3.6. The results are summarised in Table 4.2.

Table 4.2 Geotechnical data summary for the Australasian Hotel Slide.

Sample	Depth	M_C	LL	PL	PI	A	Gravel %	Sand %	Silt %	Clay %
AH40	0.8-1.2m	37	47	34	13	0.6	12	18	47	23
AH80	1.2-1.5m	42	36	26	10	0.4	3	25	47	25

Note: Clay, silt, sand and gravel fractions are weight percents.

The dominant particle size fraction was silt with roughly equal proportions of clay and sand (Table 4.2). Both of these samples are included in the non active category of Skempton (1953) despite containing swelling clays (Na Smectite, Imogolite and Swelling Chlorite). The swelling clays absorb water resulting in a decrease in soil cohesion and therefore a decrease in shear strength and in stability. To fully assess the affects that swelling clays are having on the stability requires knowledge of the percentage of each clay mineral in the sample. Quantification of the percentage of each clay mineral in the sample was beyond the scope of this thesis. Therefore, although swelling clays have been detected, the mineral percentage and therefore the effect that these are having on the stability of the slide cannot be determined. Other clay minerals detected (see Appendix A3.6) include Fe Chlorite, Mica, Kaolinite and Sepiolite.

Samples from auger holes AH40 and AH80 had low liquid limits and high plastic limits. The liquid limits for these samples are low compared with the values that were obtained in Section 3.5.2.3. These samples were collected from below the water table. Soil moisture in AH80m exceeded the liquid limit of the sample (Table 4.2). This indicates that the slide may only be marginally stable.

4.5.7 AGE AND PRESENT ACTIVITY.

The age of the landslide predates the 1943 aerial photographic run and the development of the landslide may have accompanied the deforestation of the slope in the late 1800's. However, the exact age of the landslides remains unknown. Tension cracks fresh in appearance indicate that recent movement (i.e. last 5 years) has taken place on the slide and therefore the potential for future movement and regression must exist. The appearance of fresh tension cracks is thought to relate to the discharge of storm water from the O'Sullivan house. This will be contributing to the surcharge on the slope during time periods when the slope is already receiving added surcharge from rainfall.

4.5.8 FURTHER WORK REQUIREMENTS.

Additional work is required to accurately locate the failure surface of the landslide. This would require either backhoe excavation or cored drilling. Back hoe excavation of the failure surface is unlikely to be successful as a result of the probable depth of the slide (greater than 2m) and drilling is the preferred option.

Further work is also required to assess the impact that the springs observed in the head scarp of the slide are having on the stability of the slope. These are contributing to the surcharge on the slope and are probably responsible for generating high pore pressures within the slide mass. The slide may only be marginally stable but further work is required to confirm this. The installation of piezometers along the slide surface would be useful in obtaining data on the ground water hydrology of the slide mass. This would assist any investigation of the impact that the springs have on the stability of the slope.

The emplacement of fixed survey points and monitoring these from a fixed concrete survey base station could detect active movement of the slide mass. This would be much more accurate than the method used in this investigation and would be useful in establishing a relationship between movement and precipitation. This would then allow an assessment of the stability of the slide to be made.

Control of the storm water from the O'Sullivan house is recommended if stability of the slope is to be maintained. Additional drainage measures on the slide surface should also be investigated.

4.6 DISCUSSION.

4.6.1 LANDSLIDE ASSOCIATIONS.

Landslides that have been identified in the Greymouth area can be divided into two categories based on the type of material that is involved in failure (i.e. bedrock and surficial). Landslides involving bedrock include rock block slides, rock slides, rock falls and rock avalanches. Rock slides (for example Stanton Crescent Slide) are confined to the Stillwater Mudstone and were restricted in distribution to the coastal escarpment running the length of the field area. Rock block slides, falls and avalanche type failures are confined to the Cobden Limestone and were restricted to Peter Ridge. These sorts of failures were not observed on the Twelve Apostles Range mainly because of the dense vegetation that covers the slopes. Given that they have occurred in Peter Ridge then it is likely that either they have occurred or will occur in the Twelve Apostles Range in the future.

Surficial failures include debris flows, debris or earth slides (for example the Australasian Hotel Slide) and debris or earth slumps (rotational failures). These failure types most commonly occur in colluvium and/or *in situ* regolith formed from Stillwater Mudstone (i.e. along the coastal escarpment and most other slopes in the field area). The Australasian Hotel slide involved both *in situ* regolith (layer 2 refer Figure 4.9) and colluvium (layer 1 refer Figure 4.9). Surficial failures are also found in weathering products formed from Cobden Limestone (i.e. along the Twelve Apostles Range and Peter Ridge). In general, however, the majority of surficial failures were observed in slopes that are underlain by colluvial materials. Most slopes that are underlain by colluvium have been cleared of vegetation in the past and are at present only covered by regenerating scrub. Therefore it is felt that the natural vegetation cover has an important influence in maintaining slope stability in the area by reducing the amount of precipitation reaching the ground surface and providing soil binding effects.

4.6.2 FAILURE MODELS.

Generalised failure models based on the Stanton Crescent and the Australasian Hotel landslides are presented here.

1. Rock slide (Stanton Crescent Slide).

The failure model for this type of landslide (Figure 4.12) is based on the investigation and morphology of the Stanton Crescent Slide and on the general observation of other similar failures in the field. The large "pull apart" graben that is present in these slides is suggestive of a "translational movement mechanism". A translational failure mechanism has been adopted for this landslide type, with the failure plane assumed to be bedding (Figure 4.12). Bedding within Stillwater Mudstone dipping to the west at around 20° is consistent with a bedding failure. Lateral and head releasing may be occurring along jointing within the mudstone. The graben of these slides contain detached blocks of slide material and may also be partially infilled from material that has been washed down from above. The failures range upwards from 100m in length and 100m in width and may be more than 20m in depth. However, the exact depth of these landslide could not be determined by field observation.

2. Debris or Earth Slide (Australasian Hotel Slide).

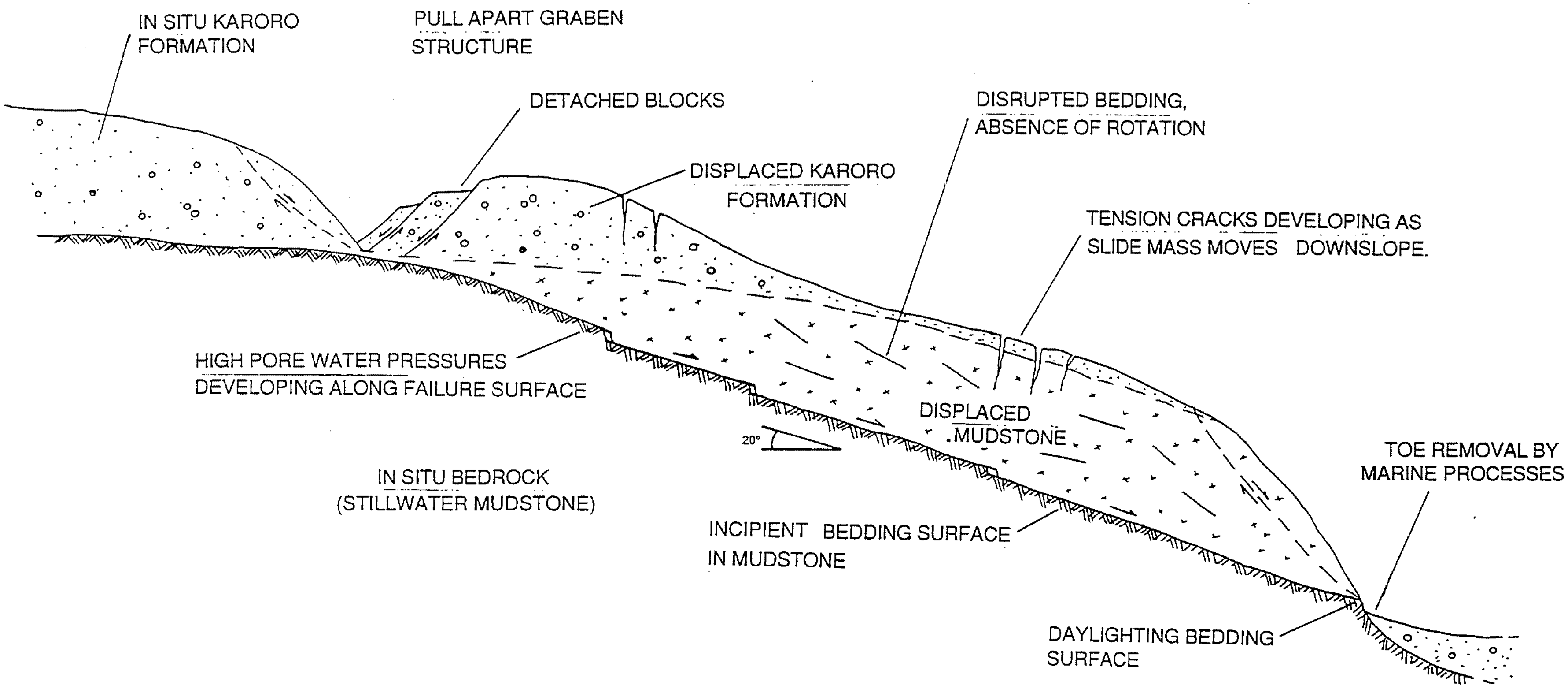
The failure model for this type of slide (Figure 4.13) is based on the investigation of the Australasian Hotel Slide and also on the general observation of other failures in the field. Debris slides have a well defined crescent head scarp area and generally also have well defined lateral margins. The failure surface for this landslide type is presumed to be the bedrock/surficial material interface, and the mode of failure is essentially translational in nature (Figure 4.13). The depth of the failure surface is dependent on the depth of alteration that has taken place at the site (generally in the range 1.5 - 3m). These failures range in size from 20-150m in length and 5-50m in width. The landslide may also undergo regression in the head scarp area as the underlying support moves down slope.

4.6.3 TRIGGERING MECHANISMS.

1. Bedrock failures.

Rock block slides and rock avalanches within the area are assumed to be generated by high ground accelerations associated with earthquakes. Rock falls are assumed to be generated by earthquakes, assisted by positive pore water pressures developing in joint systems. Slope undercutting and/or long term deterioration of the rock is also considered to cause rock falls.

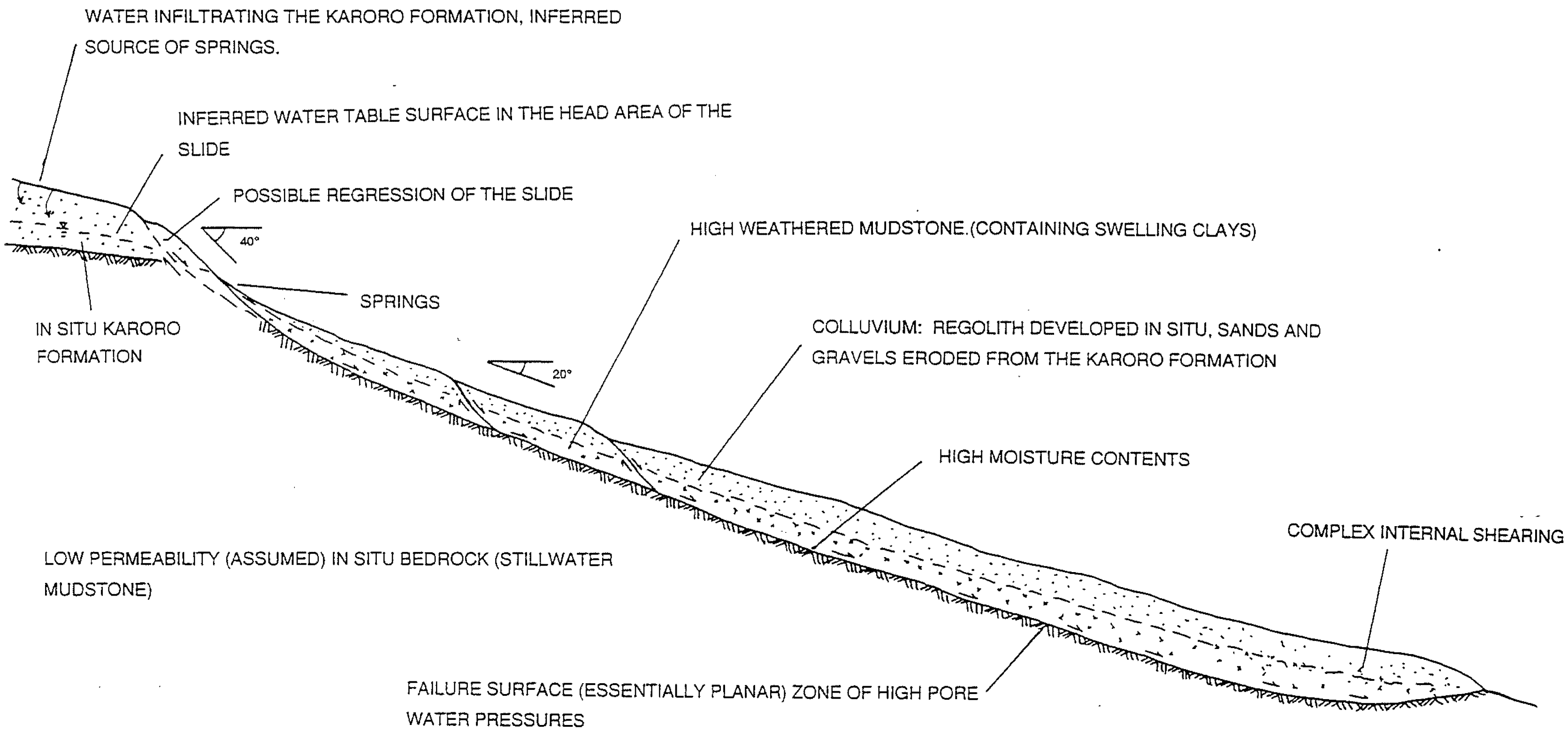
FIGURE 4.12 GENERALISED FAILURE MODEL FOR TRANSLATIONAL ROCK SLIDES.



GENERALISED TRANSLATIONAL ROCK SLIDE FAILURE MODEL, BASED ON THE INVESTIGATION OF THE STANTON CRESCENT SLIDE AND FROM THE OBSERVATION OF SIMILAR FAILURES IN THE FIELD AREA. (SEE TEXT FOR FURTHER EXPLANATION).

NOT TO SCALE

FIGURE 4.13 GENERALISED FAILURE MODEL FOR TRANSLATIONAL EARTH OR DEBRIS SLIDES.



GENERALISED TRANSLATIONAL REGOLITH FAILURE MODEL, BASED ON THE INVESTIGATION OF THE AUSTRALASIAN HOTEL SLIDE AND FROM THE OBSERVATION OF SIMILAR FAILURES IN THE FIELD AREA. (SEE TEXT FOR FURTHER EXPLANATION).

NOT TO SCALE

Rock slides (Stanton Crescent) involving the Stillwater Mudstone are assumed to have formed as a result of slope toe removal by coastal processes in conjunction with the formation of the coastal escarpment. Ground water infiltrating the rock mass and creating positive pore water pressures along an impervious surface (mud-rich failure plane) may also have contributed to the failure. These landslide types could also have been initiated by earthquakes.

2. Surficial Failures.

Water is the dominant influence on the development of landslides involving surficial materials and the effects include surcharge and a reduction in the soil cohesion. Most landslides are initiated under intense or prolonged rainfall, often in combination with other factors.

The source of the springs observed in the head scarp of the Australasian Hotel Slide is probably water that has infiltrated the Karoro Formation on the terrace surface above. These springs form part of a chain of springs that are present along the length of the coastal escarpment and are thought to be responsible for generating high uplift pressures along the surficial/bedrock material interface. The presence of these springs is considered to be the single most important factor in the development of landslides in the surficial material along the escarpment. The springs flow continuously and keep the slope in a state of semi-saturation. Failure of the soil mass may occur under added surcharge from intense or prolonged rainfall, storm water discharge associated with residential development or external disruption such as urban development or earthquakes.

Geotechnical testing has established that a component of colluvial and *in situ* samples consisted of swelling clays (Na Smectites, Swelling Chlorite and Imogolite). Swelling clays may absorb water up to 1000 percent of the original volume leading to a substantial increase in the weight of the soil. This causes positive pore water pressures with a corresponding decrease in soil cohesion. Failure of the soil mass may result when pore pressures exceed the cohesion of the soil (i.e. water content exceeds the liquid limit).

Natural soil moisture contents (M_C determined during soil characterisation) are generally close to the associated liquid limit of the sample. Given that sampling of the soil lithologies occurred following 10 days of relatively fine weather, it is probable that moisture contents exceed the liquid limit of the soil under heavy or prolonged rainfall, thereby exceeding the shear strength of the soil and resulting in failure.

Field observation revealed that in general the failure plane for these slides was the bedrock/colluvium or *in situ* regolith interface. This corresponds to the zone of highest natural moisture content within the soil profiles (determined during soil characterisation). Both Cobden Limestone and Stillwater Mudstone are assumed to have low primary (through intact rock material) permeability (secondary permeability via rock mass features however may be very high). Therefore, the implication is that high pore water pressures are generated above the bedrock surface during heavy or prolonged rainfall, thereby contributing to the instability of surficial materials.

Removal of the slope toe associated with urban development is also considered to have caused these landslides. Excavating the slope toe removes the underlying support and allows the material above to move down slope. The lack of vegetation on many of the slopes must contribute to the instability of surficial material by allowing increased precipitation to infiltrate the soil thereby causing an increase in the surcharge or loading of the soil. Shallow rooting by plants are also inferred to allow landslides to develop. The smooth nature of the bedrock interface prevents root penetration and therefore anchoring of the soil to the bedrock.

4.6.4 IMPLICATIONS FOR RESIDENTIAL DEVELOPMENT.

1. Bedrock.

Large translational rock slides in the Greymouth area have been developed for residential housing. These appear to be stable under current conditions although the effects of future development on the stability of these failures cannot be determined. The effect of uncontrolled storm water discharge associated with residential development could induce further movements on a landslide that is currently stable. In addition, storm water disposal could also initiate a first time slide. The probable cause of rock slides in Stillwater Mudstone (toe removal by coastal processes) is analogous to the removal of the slope toe during foundation excavation. The present climatic and tectonic regimes are conducive to the development of these slides and combined with urban development, could initiate new failures.

Rock falls involving the Cobden Limestone present little threat to development over much of the area. However, in the Omoto Slip area there is potential for future large rock falls similar to those that have already occurred. The Omoto Slip requires further investigation to assess the true extent of the failure and of the hazard to existing residential development.

Rock avalanches occurring in the Cobden Limestone present a high threat to development as these can travel at very high speeds and for long distances. Greymouth township is situated in the most seismically active region in New Zealand and can expect damaging earthquakes on a regular basis. It is highly probable that avalanches initiated by earthquakes will occur in the future. Those areas situated along the foot of the Twelve Apostles Range and Peter Ridge are at threat from avalanches occurring on the slopes above. Further work is clearly required to fully assess the threat presented from earthquake generated avalanches.

2. Surficial failures.

The relative frequency with which surficial materials fail means that surficial landslides present a much greater threat to urban development. Landslides occurring in surficial materials are generally areally discrete although a large number may occur over a wide area. Residential development located on slope faces are at threat from landslide occurring on the slope above. Landslides also have the potential to impact on development located along the slope toes. Slope toes are the run out areas for landslides. Housing situated on the slope crests could become involved in landsliding following landslide regression. The effects of storm water discharge or leaking water main pipes could initiate surficial landslides. Clearly there is a requirement to control the discharge of water over the terrace edges.

4.7 SYNTHESIS.

Landslides in the Greymouth area can be divided into those that involve bedrock and those involving surficial materials. In terms of the Varnes (1978) classification, the failure types that are recognised in the area are: translational rock block slides and rock slides; translational debris or earth slides; rotational debris or earth slumps; flows; falls and rock avalanches. These have been addressed in this chapter in terms of their material association and the implications that these landslides have for residential development.

Two case studies (Stanton Crescent and Australasian Hotel Slides) have been detailed and failure models for these two slides have been developed. These site specific case studies have produced information on the causes of slope instability in the Greymouth area. It was concluded that although the majority of surficial landslides are initiated under prolonged or intense rainfall, the presence of ground water springs is the single most important factor in the development of surficial landslides. The information that was obtained by these site specific studies will allow the hazard presented by these landslides to be determined (Chapter 5).

Rock slides (Stanton Crescent Slide) are likely to have failed along bedding or other similarly inclined defect. The cause of these failures has been compared with a modern day equivalent, i.e. slope toe removal associated with residential development. It was concluded that geologic, climatic and tectonic controls in the Greymouth area have the potential to generate these sorts of failure in the future and that subsequent development on currently developed failures, has the potential to initiate further movements.

CHAPTER 5.
LANDSLIDE HAZARD ZONATION.

5.1 INTRODUCTION.

A preferred approach to landslide hazard zonation for the Greymouth area, based on Horrey (1989) is developed in this chapter. The landslide hazard zonation was derived from the engineering geological data base, which in turn allowed the development of a land use capability map that is suitable for urban planning and administration. In effect, Chapter 5 represents a synthesis of the material that has been presented thus far.

Basic terminology and the principles and postulates of landslide hazard zonation are reviewed as relevant background information. The current state of the government legislation is also reviewed as pertinent back ground material, as the introduction of the Resource Management Act 1991 has provided the rational for this study. Existing landslide hazard zonation schemes in use around the world are briefly documented and these are compared with the landslide hazard zonation approach proposed here for the Greymouth area.

The underlying objective of this chapter is to develop a system that will assist both TWCRC and the Greymouth District Council in meeting their respective obligations under the RMA 1991 and the Building Act 1991. For this purpose, a brief outline of the relevant sections of each Act is provided. This chapter concludes with specific recommendations on landslide hazard zonation and outlines areas for future research.

5.2 TERMINOLOGY.

Various proponents of hazard assessment in the past have advanced and endorsed a variety of terminology, and therefore, there is a need to provide some initial clear and concise accepted nomenclature. For greater detail on the terminology the reader is referred to Varnes (1984) and Hartlen and Viberg (1988).

Natural processes become "hazards" when people and/or the environment are threatened. The identification and prediction or at least avoidance, of natural hazards are important in reducing the "risk" to human development. "Hazard mitigation" is used to reduce the effects of geological processes on people and/or the environment whereas the "avoidance option" takes account of the naturally occurring hazardous process by land-use adjustment (or zonation). The terms "hazard" and "risk" have been loosely used within the literature and have acquired a variety of definitions. Two schools of thought have developed, i.e. where: 1) hazard = event;

and 2) where hazard = the probability of occurrence. A list of common definitions (Table A4.1) for hazard and risk, including those by Varnes (1984), IPENZ (1983), BS 4779 (1979), Einstein (1988) and the RMA (1991) is given in Appendix A4.1.

Both the IPENZ (1983) and BS 4779 (1979) definitions of risk accommodate a measure of the probability of occurrence. Varnes (1984) also makes a statement on probability and introduces the terms "vulnerability" to allow for the prediction of the degree of loss, and "specific risk" which is the expected loss to a particular process. Einstein's (1988) definition of hazard is very similar to that of Varnes (1984). The definition included in the RMA (1991) does not include a statement of probability, but does make some mention of the elements at risk and in this respect is similar to Varnes (1984).

The terminology developed by Varnes (1984) is concise, widely used and accepted, and has been adopted in this study. Hazard therefore is:

> *" the probability of occurrence within a specified period of time, and within a given area, of a potentially damaging phenomena."*

Risk is:

> *"The expected degree of loss due to a particular natural phenomenon".*

The risk is the potential damage to people, property and services. Landslide hazard clearly has to be assessed before the risk can be estimated (Hutchinson (1992). Where the risk has been subdivided into specific elements the term "vulnerability" is used for *"the expected loss for a given element".* Risk assessment involves the quantification of the elements at risk and is beyond the scope of this thesis and therefore has not been attempted.

For the purpose of this report, "landslide hazard zonation" is defined as: *the division of land surface areas and the ranking of these areas according to degrees of actual or potential hazard from slope movements.*

5.3 LANDSLIDE HAZARD ASSESSMENT.

5.3.1 PRINCIPLES OF LANDSLIDE HAZARD ZONATION.

The assessment of hazardous processes (including landslides) is based on four fundamental assumptions, these being:

1. The past and present are the keys to the future. Hence natural slope failures in the future are likely to occur under geological, geomorphological and hydrological conditions that are similar to those that have led to past and present failures (Varnes 1984).
2. The main conditions that cause landsliding are controlled by physical factors and therefore are, in principle, identifiable (Hutchinson 1992).

3. The various types of landsliding can be recognised and classified both morphologically, geologically, and geotechnically (Hutchinson 1992).

4. Degrees of hazard may be estimated (Varnes 1984). In principle this is true but the extent to which the degree of hazard can be estimated is dependent on identifying and evaluating the conditions causing landsliding in 2) above (Hutchinson 1992).

In 1 above, the assumption is made that slope movements occur in similar physical settings to previous episodes of landsliding. However, it must be remembered that the absence of past or present failures does not exclude an area from landsliding in the future. Climatic change, for example, or external disruption in association with urban development, may be sufficient to induce landsliding in areas not previously susceptible. In practice, it is usual to recognise that all slopes over 15° contain a latent potential for landsliding and this principle has been adopted in this study. It should also be remembered that slopes lower than 15° may also fail.

The extent to which landslide causative factors can be identified is proportional to the level of technology used and the financial resources available. Clearly, the limiting factor in landslide assessment is the risk involved; for example, an assessment of an area low in population would not warrant the same financial commitment as a highly populated area where the risk (defined in terms of potential worth of loss) is much greater.

The difficulties of investigating a large area often necessitate the evaluation of point or site information, and extending this on a regional basis; (the approach taken in this study). This is dependent upon the accurate classification and identification of slope movement causes in the field. The extent to which the degree of hazard can be estimated (4 above) is dependent upon the identification and evaluation of the conditions causing landsliding (2 above), and on the expertise of the investigator.

5.3.2 HAZARD ZONATION.

In order to estimate degrees of hazard it is necessary to evaluate processes in terms of frequency and magnitude. Calculating the probability of occurrence of a hazardous process within a specified time interval requires a knowledge of the frequency of occurrence. The recurrence history may be obtained from historical records and/or geological study. Quantification of the probability of recurrence is usually accomplished in terms of magnitude. This is because a geological processes may occur continually and only become a hazard when a certain threshold value is exceeded.

In the simplest form, landslide hazard zonation divides the land into those areas that have the potential for sliding and those areas that do not. In more complex systems, several degrees of susceptibility may be recognised (Hartlen and Viberg 1988). This is known as "relative" slope stability and is based on data obtained through mapping.

Absolute slope stability involves either deterministic (factor of safety) or probabilistic (probability of failure) analysis and is based on calculation involving relevant geotechnical variables. Hazard zonations based on this approach require either a substantial existing engineering geological data base, or significant financial resources in order to obtain the necessary data. Empirical curves may be used where a suitable data base exists, for example, empirical relationships between slope angle and height. This analysis based approach has been used successfully in Hong Kong (Hartlen and Viberg 1988).

In all cases, the aim is to produce a landslide hazard zonation map illustrating the land surface in terms of the susceptibility to landsliding. The end use for such a map is typically land-use planning. For this purpose, the hazard zonation map is commonly converted to a "land-use capability map" delineating areas for future urban expansion.

5.3.3 ENGINEERING GEOLOGY AND LAND-USE PLANNING.

5.3.3.1 Legislative Framework.

Prior to 1991, urban development in New Zealand was controlled by five pieces of parliamentary legislation (see Table 5.1). These, however, have now largely been superseded by the introduction of the Resource Management Act 1991 and the Building Act 1991, although a much modified version of the Local Government Act 1974 remains.

The functions of Regional Councils under the "Regional Plan" are as follows, under:

"Section 30. Functions of the regional councils under this act -
(1)(c) *The control of the use of land for the purpose of ....*
(iv) *The avoidance or mitigation of natural hazards ....*

and Section 35. Duty to gather information, monitor and keep records -
(5) *The information to be kept by the local authority ... shall include ..*
(j) *Records of natural hazards to the extent that the local authority considers appropriate for the effective discharge of its functions ..."*

Territorial authorities also have statutory requirements under the RMA 1991. These include, under Section 31, the duty to implement "*rules for the avoidance or mitigation of natural hazards*" and "*the control of subdivision of land*" in the formation of District Plans. The Building Act 1991 controls building approval by way of a consent. Territorial authorities under Section 36 (1)(a), (b) of the Building Act 1991 are required to refuse a building consent if:

a) *The land on which the building work is to take place is subject to, or likely to be subject to erosion, avulsion, alluvion, falling debris, subsidence, inundation, or slippage; or*

b) *The building work itself is likely to accelerate, worsen, or result in erosion, avulsion, alluvion, falling debris, subsidence, inundation or slippage of that land or any other property-*

Table 5.1

URBAN PLANNING LEGISLATION IN NEW ZEALAND prior to 1991[1]

Principal Act[2]	Planning Function	Comments
A. *TOWN AND COUNTRY PLANNING ACT 1977*	1. *Regional Schemes*	Designed for wise use of resources and implementation of long-term "regional" planning policies; establishment of regional or united councils
	2. *District Schemes*	Detailed planning and admininstration of local "districts"; territorial authorities with responsibility for land-use and development practices
	3. *Maritime Schemes*	Preservation and conservation function in addition to establishment and administration of maritime facilities such as harbours
B. *LOCAL GOVERNMENT ACT 1974*	1. *Scheme Plans*	Local councils have responsibility for urban subdivision codes and preparation of scheme plans for residential development
	2. *By-Law Control*	Local councils also responsible for building permit approval, the adequacy of building sites, and control of earthworks or land instability
	3. *Provision of Services*	Functions of local authorities include roading, water supply, stormwater and sewerage disposal, etc
C. *SOIL CONSERVATION AND RIVERS CONTROL ACT 1941*	1. *Catchment Authorities*	Required to deal with river and erosion control, soil conservation and drainage works, including flood mitigation
	2. *Erosion Control*	Statutory powers available to control earthworks that may result in erosion or siltation
D. *WATER AND SOIL CONSERVATION ACT 1967*	1. *Regional Water Boards*	Established to control access to surface and underground water, including waste disposal and pollution
	2. *National Water and Soil Conservation Authority (NWASCO)*	Established as national coordinating authority, with research functions in areas such as erosion assessment
E. *EARTHQUAKE AND WAR DAMAGE ACT 1944*	1. *Earthquake Insurance*	Provision for compensation for earthquake shock and resultant fire damage to property
	2. *Disaster Fund*	Amendment to provide for damage due to abnormal storm, flood or volcanic eruption
	3. *Landslip Insurance*	Amendment to provide automatic cover for property damage from landslip but <u>not</u> from settlement, soil shrinkage or compaction
	4. *Geothermal Activity*	Cover available on a voluntary basis only

NOTES: 1) Table based on Sheppard (1983) and Gill (1974).

2) A,B,C & D now superceded by Resource Management Act 1991 and Building Act 1991.

Territorial authorities therefore are required not only to consider the land being subdivided but also the effects of subdivision on the stability of adjacent properties. However, Section 36 of the Building Act 1991 allows for the development of land that may be subjected to either a) or b) above, provided that the land owner assumes liability which then becomes attached to the land title and is transferred during sale.

5.3.3.2 Engineering geology data input.

The engineering geological methodology developed by Bell and Pettinga (1984) for the purpose of hazard assessment and subdivision planning has been adopted in this assessment of the Greymouth area. The various levels of engineering geological input that may be required for urban planning are illustrated in Table 5.2. This study has presented maps at a scale of 1:10 000 and therefore is included within the "District Plan" stage of urban development. "District Plan" has been defined under the RMA 1991 and replaces the "District Scheme" term used in the Local Government Act 1974.

Under the Bell and Pettinga (1984) approach, site investigation includes aerial photographic interpretation, field mapping, exposure logging and a geotechnical appraisal of sampled materials (Chapter 3). This information is then integrated as a "site model" (Chapter 4) allowing the identification of areas requiring additional investigation, and those areas that are unsuited to urban development (Bell and Pettinga 1985).

5.4 A REVIEW OF HAZARD ZONATION SCHEMES CURRENTLY IN USE.

5.4.1 GENERAL.

Hazard zonation schemes used both here in New Zealand and overseas (relative slope stability -California, Geotechnical Area Studies Programme (GASP) - Hong Kong, ZERMOS - France, Pattern-Unit-Component-Evaluation (PUCE) - Australia, and Urban Land Use Capability (ULUC) - New Zealand) are reviewed in detail in Appendix A4.2 and are briefly outlined below.

In the relative slope stability approach used by Brabb *et al.* (1972) for the San Mateo County - California, landslide susceptibility was estimated from a weighted combination of landslide distribution, slope angles and bedrock materials. The various land units identified were then numerically ranked according to susceptibility to landsliding (see Table A4.2).

Table 5.2

ENGINEERING GEOLOGY DATA INPUT FOR URBAN DEVELOPMENT IN NEW ZEALAND

Planning Stage (1)	Engineering Geology (2) Investigation Objectives	Typical Map Scales	Geotechnical Data (3)
A. REGIONAL SCHEME	1. Identification of "regional" hazards such as floodplains and "active" fault traces 2. Mapping of bedrock and surficial geology	1:100,000 ↓ 1:25,000	Characterisation of lithologies and identification of "problem" soil types; assessment of resources (e.g. aggregate availability and long-term requirements)
B. DISTRICT SCHEME	1. Engineering geological and/or pedological mapping, with limited excavation logging 2. Identification and investigation of "local" hazards (e.g. landslides)	1:10,000 ↓ 1:5,000	Geotechnical characterisation of mapping units as required for land-use zoning decisions; specific evaluation of tectonic and hydrologic hazards
C. SUBDIVISION CONCEPT PLAN	1. Engineering geological site mapping and subsurface investigations 2. Interpretative risk assessment and/or planning guidelines	1:2,000 ↓ 1:1,000	Limited testing (e.g. plasticity/grainsize) to indicate general characteristics of site materials; hazard avoidance or mitigation measures
D. SUBDIVISION SCHEME PLANS	1. Detailed site investigation of specific areas identified at Concept Plan stage 2. Engineering geological mapping and logging to meet any "local" authority requirements	1:1,000 ↓ 1:500	Additional geotechnical testing to verify design and/or construction feasibility as required; investigation of specific features to facilitate stage E
E. SUBDIVISION DESIGN AND CONSTRUCTION	1. Confirmation of mapped geology 2. Additional investigation as required	1:500 ↓ 1:50	Detailed investigations for design of cut and fill batters if required; control of earthworks
F. SECTION DEVELOPMENT AND HOUSE CONSTRUCTION	Engineering geological investigations only if required (A + E should prevent site "problems")	1:200 ↓ 1:50	Site specific testing for foundations if required; control of earthworks, drainage, etc

NOTES: 1) Planning stages follow from existing legislative framework (Table 3).

2) Engineering geology investigation methods include air-photo interpretation and relevant mapping and logging techniques.

3) Geotechnical design investigation requirements may vary considerably within individual urban areas.

The GASP is based on a terrain evaluation approach (aerial photographic interpretation, site reconnaissance and existing information). Various GASP maps are produced from which an interpretative GLUM (Geotechnical Land Use Map) is then developed for specific areas. This divides the land into 4 categories (see Table A4.5) dependent on the limitations and suitability for development.

The ZERMOS system relies heavily on processes mapping. Landslide orientated geomorphological maps showing all existing landslides and landforms of an area, plus any geological and hydrological information are compiled. Then interpretative 1:25 000 maps in which a six-class zoning approach is used for active and potential slope movements are derived.

Terrain evaluation forms the basis of the PUCE system. The total area mapped is divided into a number of basic terrain units of similar morphology, geology and landslide characteristics. From this data base, hazardous processes can be analysed and hazard maps compiled accordingly.

ULUC surveys involve mapping inventory data on geology, soils, slopes, vegetation, erosion, drainage, existing land use and terrain. This information is then integrated and a subjective land use capability class (see Table A4.6) is assigned to a recognised, discrete area.

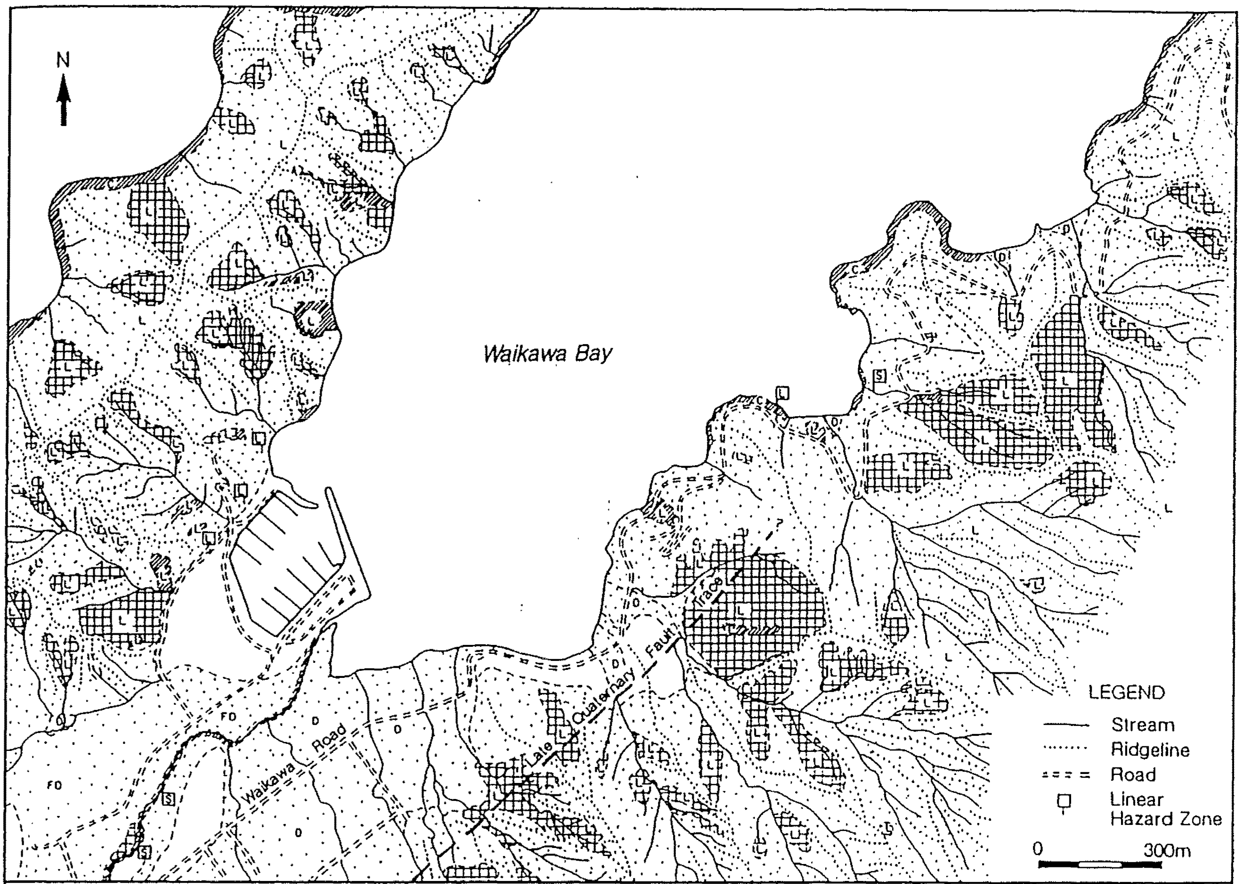
The recommended approach to landslide hazard zonation developed for the Greymouth area closely follows an approach used by Horrey (1989) in the Marlborough Sounds. Horrey (1989) assessed hazardous processes including landslides, flooding, seismicity and coastal erosion for the Picton, Waikawa and Shakespeare Bay areas. This data was integrated to form hazard maps (at a scale of 1;10 000 and 1:5000) comprised of four hazard classes that were based on the estimated age of the most recent activity (Figure 5.1). A development suitability map was derived from the hazard map (Figure 5.2). This consisted of four development suitability classes representing increasing geotechnical limitations to development (Figure 5.2).

5.4.2 DISCUSSION.

The following discussion is largely based on Horrey (1989) who reviewed these schemes. In regard to the five hazard zonation schemes documented above, the following points can be made.

1) The scale of investigation dictates the approach to hazard zonation that is taken. Mapping at a scale of 1:60 000 to 1:125 000 as in California necessitates the adoption of broad scale map units. This contrasts with the approach taken in Hong Kong where 1:2500 scale mapping allows much greater detail to be obtained. The rigid system of terrain evaluation used in ULUC is regarded as a weakness as it does not allow for increased detail at smaller scales.

2) Financial resources and the quality of existing information are the primary constraint on the detail and accuracy of the hazard assessment system.

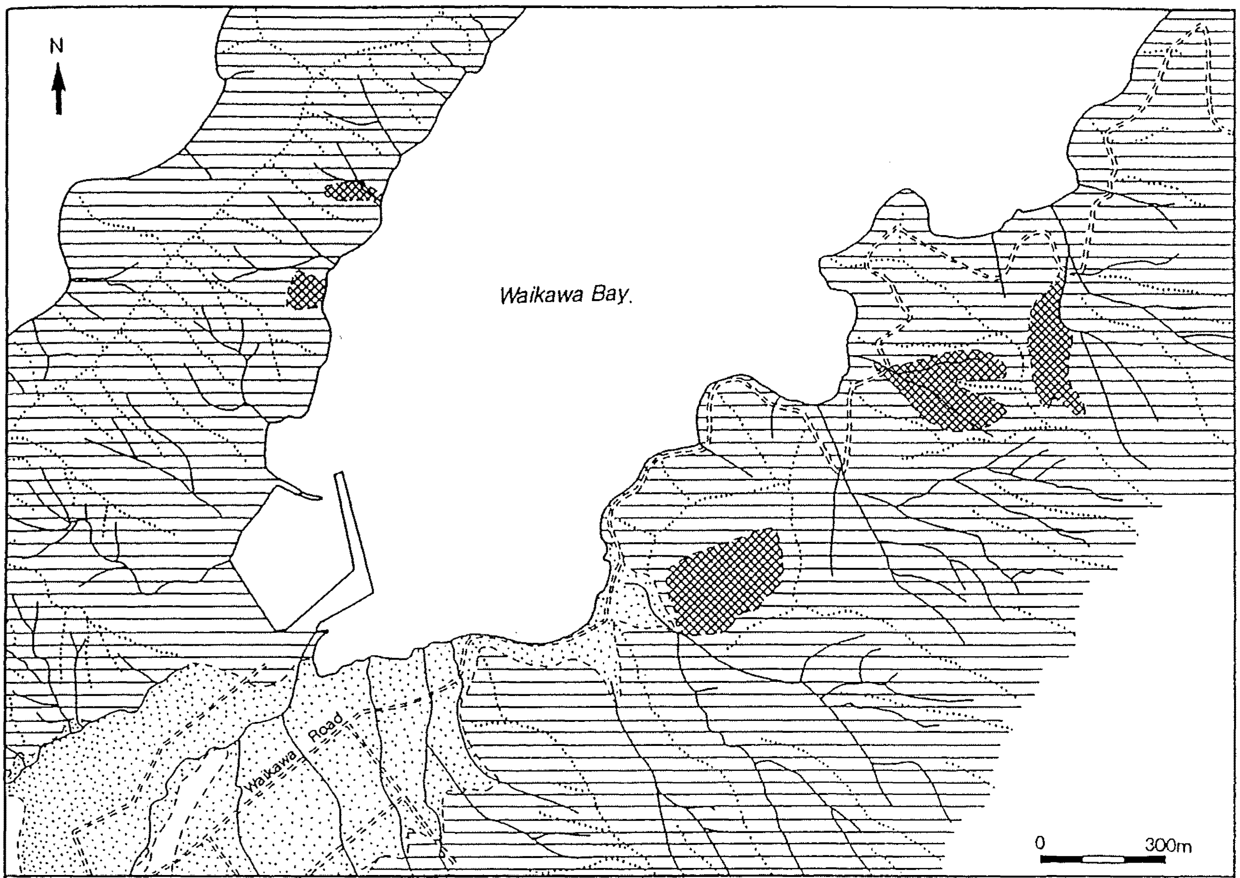


HAZARD ZONE DEFINITION

PROCESS / DEGREE OF HAZARD	Slope Movement L	Flooding F	Debris Deposition D	Stream Bank Erosion S	Coastal Erosion C	Ground Subsidence W
High	Slope movements active during the last 30 yrs	Areas liable to inundation in a 30 yr. period, assuming adequate performance of existing flood protection works	Areas liable to debris deposition, siltation, and channel migration in a 30 yr. period	Sites of active removal of material at the stream bank within the last 30 yrs	Sites of active removal of material at the shoreline within the last 30 yrs	Areas of permanent ponded drainage. Potential ground subsidence hazard
Moderate	Slope movements showing no sign of activity during the last 30 yrs.	Areas liable to inundation in a 100 yr. period, assuming adequate performance of existing flood protection works	Areas liable to debris deposition, siltation, and channel migration in a 100 yr. period	Insufficient data available to assign low and moderate hazard		
Low	Slope angle in excess of 15°, with no history of slope movement identified	Areas liable to inundation in an extreme flood event, assuming failure of existing flood protection works	Areas liable to debris deposition, siltation, and channel migration in an extreme rainfall event			
Negligible	Slope angle less than 15°, with no history of slope movement identified	No significant hazard	No significant hazard	No significant hazard	No significant hazard	No significant hazard

Not shown on map

Figure 5.1 Part of the hazard zonation map with legend for the Picton-Waikawa area (redrawn from Horrey 1989).



PROCESS / DEGREE OF HAZARD	Slope Movement	Flooding	Debris Deposition	Stream Bank Erosion	Coastal Erosion	Ground Subsidence
HIGH Class IV	Extreme limitations. Development not recommended	Extreme limitations. Development not recommended.	Extreme limitations. Development not recommended.	Development not recommended without certified engineering design of stream bank protection works.	Development not recommended without certified engineering design of coastal protection works.	Unsuitable for development without site de-watering and filling. Further investigation of ground subsidence potential required.
MODERATE Class III	Significant limitations. detailed site investigations required. Development may be possible on selected sites.	Significant limitations. Site investigation of flood hazard required. Protective measures and special foundation design may be required.	Significant limitations. Site investigation of debris and siltation hazard required. Culvert and watercourse design critical.	Insufficient data available to assign low and moderate hazard		
LOW Class II	Some limitations. Engineering design of foundations and stormwater and effluent disposal required.	Some limitations. Generally favourable, but some assessment of flood hazard may be required. Stormwater control important.	Some limitations. Generally favourable, but some assessment of debris and siltation hazard may be required. Culvert and watercourse design important.			
NEGLIGIBLE Class I	Suitable for residential development given normal engineering prudence, and compliance with local authority by-laws. Not shown on map					

Figure 5.2 Part of the development suitability map and classification for the Picton-Waikawa area (redrawn from Horrey 1989).

3) Overseas systems require a high level of basic engineering geological data. These systems would not be applicable where such a data base is unavailable. Published information within New Zealand includes broad to medium scale geological and pedological maps but these lack engineering geological detail, particularly of surficial map units.

4) Systems based on surface morphology such as the PUCE and ULUC systems do not place enough emphasis on the engineering geological properties of the mapped geology. In addition these systems fail to make the distinction between active or potentially active geological processes.

Horrey (1989) concluded that although terrain evaluation is a convenient method of acquiring and storing data, it is necessary to assess the active geological processes in order to obtain some appreciation of the hazard involved. This is because terrain evaluation is based on the present situation in a landscape and therefore may neglect the dynamic and evolutionary aspects of the area (Hutchinson 1992).

5.5 LANDSLIDE HAZARDS IN THE GREYMOUTH AREA.

Landslides in the Greymouth area have been a common occurrence in the geomorphological development of the landscape. Most landslides have taken place well away from urban development and only recently as urban expansion has impinged on the hill country has a significant landslide hazard become apparent.

A history of landslides in chronological order to have affected Greymouth since European settlement is given in Appendix A4.3. This was compiled from various newspaper reports and from an existing data base in TWCRC. The record is likely to be incomplete for several reasons, including: 1) the early reports were highly subjective and often inconsistent; 2) the historical record is incomplete as some records have been destroyed either by fire or flooding; and 3) many landslides occur in unpopulated areas and go unrecorded. The record shows that the majority of landslides that have occurred in the Greymouth area follow prolonged or intense rainfall. These failures appear to involve surficial materials and newspaper reports suggest that they are in the flow, debris or earth slide, or slump category of Varnes (1978). Earthquakes have also triggered landslides in the area and these are falls involving the Cobden Limestone and slides involving colluvium in the Omoto Slip area.

Identified landslide types within the mapped area have been classified on the basis of the mode of failure and by the materials involved in sliding. Failure types and causative (both triggering and pre-existing) factors have been fully discussed in Chapters 2 and 4. From a close examination of landslides in the field and from the historic data base it would appear that intense or prolonged rainfall is the primary agent causing landsliding. As in other areas, for example Wellington (Crozier and Eyles 1980), it is probable that there is a rainfall threshold value, which, if exceeded will result in failure. To establish the threshold value, detailed information on the antecedent soil moisture contents and records of the frequency of landsliding and rainfall intensity are required.

From an analysis of the landslide records (Appendix A4.3), it is apparent that landsliding may result from as little as 45mm of rainfall in a 24 hour period which equates to a 1 hour duration storm with a return period of 5 years (see Table 1.1). However, landsliding may only occur after much heavier rainfall, for example 169mm/24 hours, which equates to a 24 hour duration storm with a return period of 10 years. Clearly, the antecedent soil moisture history is an important factor in landslide development under varying rainfall intensities and frequencies in the Greymouth area. The lack of comprehensive data on the relationship between soil moisture history, rainfall intensity and landslide development precludes developing a frequency-magnitude hazard zonation approach for landsliding in the Greymouth area.

Geotechnical testing (discussed in Chapter 3) has established a guide to the expected foundation conditions for surficial and bedrock lithologies within the area. These are, however, likely to show considerable variation over the area and in addition, the subsurface hydrogeological conditions are almost unknown. Therefore, the use of deterministic (i.e. based on factor of safety) methods of hazard zonation are inappropriate in this case.

5.6 PROPOSED LANDSLIDE HAZARD SCHEME.

5.6.1 CURRENT LAND-USE ZONING PRACTICES IN THE GREYMOUTH BOROUGH AREA.

Urban development within the Greymouth Borough area, including residential, industrial and associated engineering works, is presently controlled by the Greymouth Borough District Scheme which was last reviewed in 1987. The problems of land instability have been previously addressed in a report entitled "Land Use Capability Surveys of Greymouth and Survey County Town" commissioned by the now defunct Westland Catchment Board. The report was compiled by Hutchison and McKie (1979) (reviewed in Appendix A4.2) and has been incorporated into the Greymouth Borough District Scheme as six zones outlining the suitability for urban development in terms of susceptibility to soil erosion and flooding (Figure 5.3).

At the time of subdivision approval under the District Scheme and where there is a perceived threat of land instability, the Borough Engineer may:

> *"... require tests to be carried out and reports prepared for any proposed subdivision, to determine and analyse the sub-soil conditions with a view to ensuring soil stability during the after development, and to controlling the effects of excessive run off during heavy rain. Such tests shall be carried out under the supervision of a registered engineer or other suitably qualified person who shall sign a statement that the works proposed are satisfactory in regard to stability and that the proposed sites are suitable for building."*

This applies to the land over 12° in slope at which time the subdivision proposer requires a resource consent under Section 34 of the Soil and Conservation Act 1941 which is issued by the local regional authority. (It should be noted at this stage, that this study adopts a slope angle of 15° above which there is assumed some potential for landslides to develop.) Section 641 of the Local Government Act 1974:

"... *prohibits the council from issuing a building permit unless satisfied that provision has been made or will be for the protection of the land. These considerations of land stability will over-rule any zonings of the land shown to be subjected to a hazard only when adequate protective works are proposed either on a collective or community basis or on an individual basis.*"

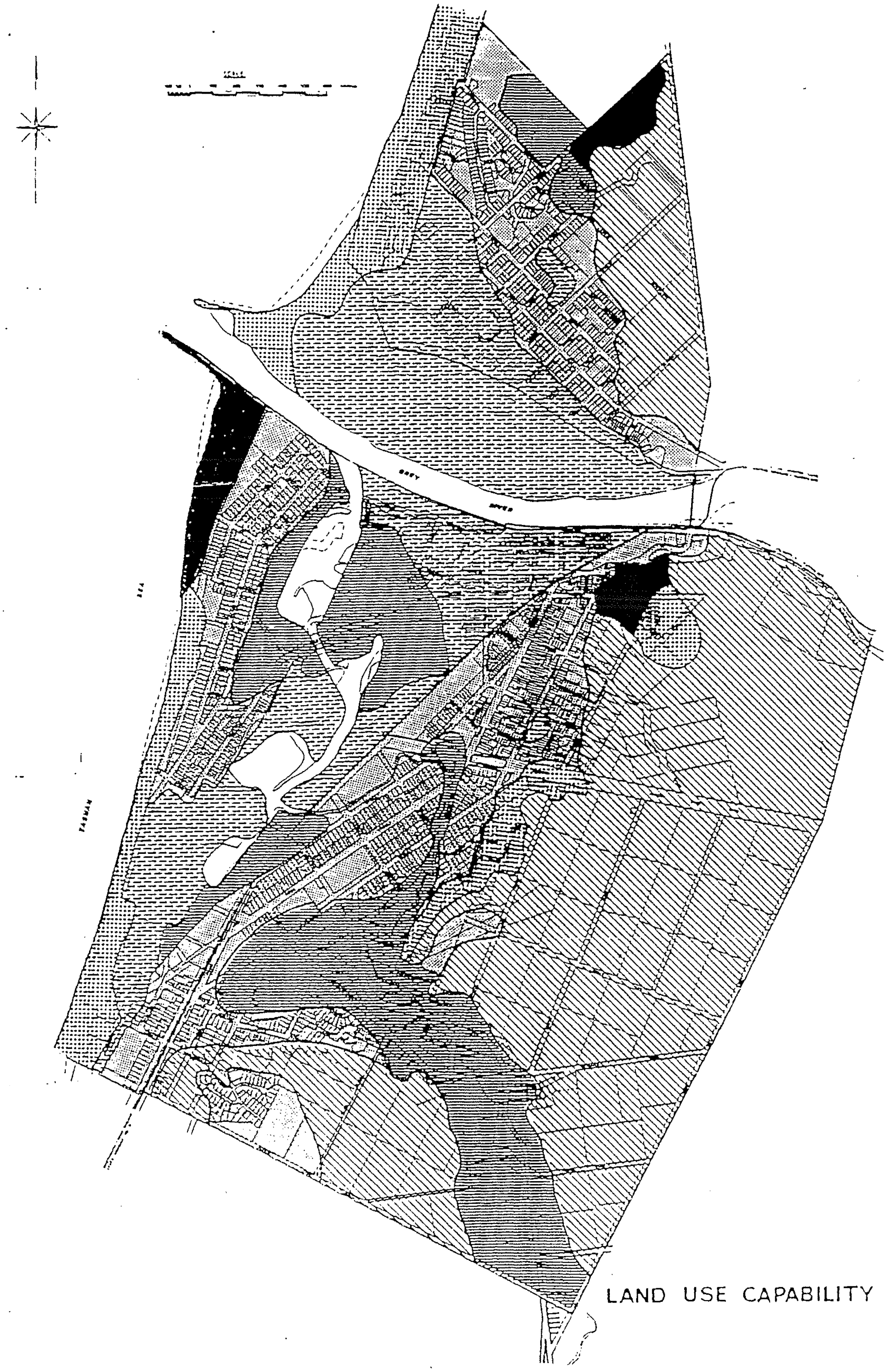
The Greymouth Borough District Scheme was developed under the Town and Country Planning Act 1977. As indicated earlier, the RMA 1991 and the Building Act 1991 have replaced the older legislation. The Greymouth District Council are currently in a transitional phase and are formulating a "District Plan" in accordance with the new legislation.

5.6.2 LAND-USE ZONING, WESTLAND COUNTY.

Areas outside the Greymouth Borough were controlled by the Grey County Council whose Ordinance V, clause (1) states:

"(1) <u>Land to be suitable for proposed use.</u> *Notwithstanding conformity with the zoning requirements of these Ordinances, no building shall be erected or placed and no use shall be established on any land which is not suitable for the use proposed, and for the purpose of determining whether any land is suitable for any particular use, regard shall be had to the best use of the land and its economic servicing and development to earthquake fault lines, to liability to flooding, erosion or landslip, to stability of foundations and to safety, health and amenities.*"

The Grey County Council has been amalgamated with the Greymouth Borough Council and the Runanga County Council to become the Greymouth District Council. Planning and subdivision is now a function of this Territorial Authority under the RMA 1991. The District Plan under review by the Greymouth District Council is due for completion and implementation later this year.



ZONE | DESCRIPTION

- Areas suitable for all urban uses including residential, commercial and industrial uses based on soil types, erosion rating and flooding potential.
- Areas with slight soil or erosion limitations to urban development or use. Development of these areas is recommended only after their inherent soil or erosion problems have been overcome.
- Areas with severe soil or erosion limitations to urban development or use. Development of these areas should only proceed with extreme caution and following engineering recommendations.
- Areas with slight flooding limitations to urban development or use. Development of these areas is recommended only after the flooding problems have been overcome and the risks minimised.
- Areas of severe flooding potential on the floodplain of the Grey River. Further development of this zone should only proceed after adequate flood protection works have been completed.
- Areas not recommended for any form of urban development because of soil erosion and stability both on-site and off-site. Urban development should only proceed on sound engineering advice based on extensive site specific tests and provided that access is satisfactory and off-site problems will not occur.

Figure 5.3 Existing land use zoning with legend for the Greymouth urban area (from Greymouth Borough District Scheme 1987).

5.6.3 RECOMMENDED LANDSLIDE HAZARD ZONATION FOR GREYMOUTH.

5.6.3.1 Engineering geological mapping - applications and limitations.

The approach taken in mapping the field area has been fully discussed in Chapter 3. The material obtained by field mapping and aerial photographic interpretation together with information obtained from a study of historical records (Appendix A4.3) is shown in Figures 3.3a,b,c. These three maps represent an engineering geological data base from which a hazard zonation map has been compiled.

The age of landslide activity was estimated from a series of aerial photographs that span the last 50 years. Form these, it was possible to develop a three fold age classification for identified failures. Each landslide has been annotated with a numerical code based on the age of the failure (1 = <5 years, 2 = 5 to 50 years, and 3 = >50 years). Many of the landslides identified are simply too small to be represented at the scale of mapping used. These have been represented as a point source (see Figure 3.3a,b,c). To illustrate the materials involved in landsliding, identified failures have been annotated as S = surficial material and B = bedrock.

The engineering geological information is displayed on 3 contour maps (contour interval 20m) at a scale of 1:10 000 (Figures 3.3a,b,c). Geological units are illustrated with a series of colours based on the age of the deposit. Regolith deposits have been illustrated in Figure 2.2 and it was felt unnecessary to reproduce the information. Morphological features such as scarps have been illustrated using graphic symbols.

The primary use of these maps is for the development of hazard zonation maps. The broad scale used in mapping means that planners should regard these as a guide to the expected foundations conditions. These maps are not intended to be a substitute for site specific investigations, for example carried out at the subdivision planning or section development stage and <u>must never</u> be considered as such.

5.6.3.2 Hazard zonation mapping - applications and limitations.

The objective of the hazard zonation maps of the Greymouth area (Figures 5.4a,b,c) are to show the areal extent of the relative hazard (either potential or existing) from landslides. Areas of landsliding were transferred directly to the hazard maps. The degree of hazard could then be estimated from the age of the most recent activity (Figure 5.5). The age of activity falls into the following classes: 0 - 50 year period (representing aerial photographic coverage); 50 - 120 year period (representing historical records), and greater than 120 years (evidence obtained from the interpretation of the geological record).

Although landsliding is likely to occur in areas that have suffered instability in the past, first time slides may occur in areas previously unaffected by landslide due to changing conditions (for example urban development). Recognising this, and based on the assumption that all slopes over 15° have some potential for landsliding, all slopes have been given some degree of hazard. A slope angle of 15° is regarded as the lower bound for landslide to develop in the Greymouth area. Areas that have suffered instability in the last 50 years were included in the high hazard category, whereas those areas showing no sign of activity in the last 50 years but have suffered instability in the past are included in the medium hazard category (Figure 5.5). Slopes with angles in excess of 15° but exhibiting no sign of instability are included in the low hazard category. Areas in which the slope angle is less than 15° and show no sign of landslide activity are included in the negligible hazard category (Figure 5.5).

The hazard maps are presented at a scale of 1:10 000 and the degrees of hazard are illustrated using a traffic-light colour coding system, red for high hazard and green for low hazard. Four classes of hazard are recognised (Figure 5.5) based on the potential for sliding.

The landslide hazard maps are intended as a guide for planners as to the level of future investigation required in each area. These hazard maps are based on the existing or perceived landslide hazard but "do not predict" the effects of future urban development on the landscape. As the landslide hazard maps are derived from the engineering geological map and because of problems with dense vegetation and site access (outlined previously), these maps should be used with some caution.

HAZARD ZONATION CLASSES

☐	HIGH	Slope movements occurring within the last 50 years.
☐	MEDIUM	Slope movements showing no sign of activity in the last 50 years.
☐	LOW	Slope angle in excess of 15° with no history of slope movements.
☐	NEGLIGIBLE	Slope angle less than 15° no history of slope instability.

Figure 5.5 Hazard classes used in the hazard zonation of the field area.

5.6.3.3 Development suitability maps.

Land use suitability maps (Figures 5.6a,b,c) have been compiled at a scale of 1:10 000 from the hazard zonation maps and these represent the end stage in the landslide hazard zonation process. The maps are intended for land use planning and it is expected that the Greymouth District Council will incorporate these into the District Plan. Land use capability maps differ from the engineering geological and hazard zonation maps in that they do not show the extent of landslides, only the suitability of the land for development.

The mapped area has been divided into four categories based on the suitability for development (Figure 5.7) although the classes that have been adopted may not necessarily correspond to the landslide hazard zonation classes. Class I land represents the areas most suitable for all types of development and includes the coastal plain, river flats and the terrace surfaces (Figure 5.7). Class II land is suitable for urban development and only contains slight limitations to development. Some additional site investigation may be required (Figure 5.7). Class III land contains moderate geotechnical limitations to development and includes all slopes over 15°. A detailed site investigation should accompany development and specially designed foundations may be required. Class IV land represents very high geotechnical constraints to development and these areas are not considered suitable. Development should only proceed following an intensive site investigation programme.

DEVELOPMENT SUITABILITY CLASSES

CLASS IV	Extreme geotechnical limitations to development. Generally unsuitable.
CLASS III	Moderate geotechnical limitations to devlopment. Additional detailed site investigation is required.
CLASS II	Low geotechnical limitations to development. Generally suitable although some additional site investigation may be required.
CLASS I	No significant geotechnical constraints. Suitable for all types of development.

Figure 5.7 The four development suitability land classes used in the development suitability maps of the field area.

The land use capability maps are somewhat subjective in that the boundaries have been adjusted to include land that is not presently involved in landsliding, but which is similar in physical setting to land that is or has been involved in landsliding. These maps also contain the same limitations as both of the engineering geological and the landslide hazard zonation maps. It is felt that as these maps have been based on sound engineering geological information, they therefore represent a reasonable guide to areas that are suitable for future urban development and in addition, outline the expected degree of geotechnical investigation that may be required in each area.

5.6.4 COMPARISON WITH OTHER SCHEMES.

The differences in scale between this study and the Zermos, PUCE and California systems limit the comparisons that can be made. At large scales (1:20 000 or greater) the use of remote sensing is more economically justifiable than the approach taken in this study. However, some form of field checking is required to assess the accuracy of the system prior to use in urban planning.

The methodology in this system is similar to that of the ULUC system although this system, is based on the engineering geological aspects while the ULUC system is based upon soil conservation principles. ULUC assessments are best suited to broad scale applications and Bell (1987) has questioned the use of such systems at small scales (1:5000). Detailed terrain assessments require sound engineering geological information, as used in the GASP programme in Hong Kong, to be effective.

The system proposed for the Greymouth area is analogous to the GASP programme in Hong Kong. The engineering geological map used in this study represents the initial data base in the GASP approach and the hazard zonation map is equivalent to the physical constraints map in that they both define the type of hazardous processes affecting a particular area (even though this study is only concerned with landslides). The development suitability map has the same function as the GLUM of the GASP approach, although again it must be remembered that this study is only concerned with landsliding. Clearly there is a vast difference in the financial resources available to each study and this is reflected in the prerequisite for detailed geotechnical data required in the GASP system.

The landslide hazard zonation is closely aligned with that used by Horrey (1989) in the Marlborough Sounds area. The fundamental difference in the studies is that while Horrey (1989) assessed a range of hazardous processes, this study has concentrated solely on landslides. Both studies have used similar scales (i.e. 1:10 000) although Horrey (1989) also used a scale of 1:5000 in certain areas (Waikawa Bay). In both systems, the hazard zonation classes that were used are very similar. However, while Horrey (1989) defines a high hazard category as an area involved in landsliding in the last 30 years, this study recognises a high hazard category for slope movements occurring in an area in the last 50 years. In part this results from the longer period of aerial photographic coverage that is available in the Greymouth area, but also recognises the highly active and evolutionary nature of the Greymouth environment. Despite differences, it remains that both studies have adopted an approach to the assessment of natural hazards that is based on an engineering geological appraisal of active geological processes in the area under study and have produced hazard zonation and land-use capability maps that are suitable for urban planning and administration.

This study has been completed at a minimum cost, without the need for expensive equipment or sophisticated computer analysis that is required under some of the other systems. The hazard zonation map has been based on an engineering geological assessment of active landslide processes and should provide an accurate guide for future planners.

5.6.5 RECOMMENDATIONS.

The landslide hazard zonation scheme developed for the Greymouth urban area has been specifically designed with urban planning in mind. Therefore it is anticipated that the Greymouth District Council will incorporate the data generated by this study within the District Plan (currently being revised), thereby avoiding (or at least reducing) the landslide hazard to future urban development.

This study should assist The West Coast Regional Council in meeting its statutory requirements (with respect to landslides) under Section 30(1)(c)(iv) and 35(5)(j) of the Resource Management Act 1991 and will assist the Greymouth District Council in meeting its statutory requirements (with respect to landslides) under Section 31(b) of the Resource Management Act, and Section 36(1)(a), (b) of the Building Act 1991.

It is expected that this landslide hazard zonation scheme will be used in conjunction with other hazard zonation systems developed for different processes, for example flooding. Therefore some of the areas zoned as low hazard to landslides may be included within a high hazard category from other processes.

It should also be noted that the landslide hazard zonation and land-use suitability maps provide a "guide" both to the expected foundation conditions and level of site investigation that may be required in a given area. These maps should <u>never</u> be used as a substitute for site specific investigation.

In areas with a perceived landslide hazard to future development (i.e. Classes II, III, and IV in the development Suitability Map) it is recommended that the Council require an engineering geological plan to be submitted at a suitable scale prior to issuing a building consent. This plan is to be completed by a person qualified in engineering geology or some other suitability qualified person, for the purpose of certifying the proposed subdivision safe for development.

Control of the storm water both along the slope face and on the terrace surface above is of paramount importance in maintaining stability of the slopes. In particular, restrictions should be placed upon the uncontrolled discharge of water over the terrace edge. In addition, removal or excavation of the slope toe (particularly within Stillwater Mudstone) is not recommended. This is analogous to the marine cliffing processes that has formed a series of large translational bedrock failures (Stanton Crescent) along the coastal escarpment. Hence the potential must exist for similar failures to occur following excavation of the slope toe in the future. It is recommended that a zone free from development is maintained along the slope toes. This would minimise the potential hazard from the "run out" of landslides occurring on the slopes above. Further work is required in this area to quantify both the size of the landslides that may occur, and also of the expected speed and distance with which they would travel.

The importance of rainfall in initiating landslides has been emphasised thus far. Clearly further work is required to establish soil moisture threshold values above which landsliding may take place in relation to rainfall magnitude and frequency within the area. Further investigation is also required to be carried out into the hazard posed by earthquake generated failures.

This landslide assessment is designed to be the beginning of a series that will be carried out in the West Coast region with the purpose of identifying areas suitable for future development. It is also recommended that council reviews and updates this assessment on a regular basis (for example every 10 years) in order to accommodate change (for example climatic or land-use) in the area.

5.6.6 SYNTHESIS.

A landslide hazard zonation of the study area has identified suitable areas for future urban development. This was based on the engineering geological investigation of the area combined with aerial photographic interpretation, geotechnical evaluation and an analysis of historical records. The land use suitability maps are based on an element of subjectivity and should be used with some caution. It is expected that as additional material becomes available, the hazard zonation will be updated. The history of landslides that have affected the Greymouth area (presented in Appendix A5.2) will assist TWCRC under Section 30 and 35 of the RMA 1991 with respect to landslides. In addition, this study should assist the Greymouth District Council under Section 31 of the RMA 1991 and Section 36 of the Building Act 1991.

CHAPTER 6.
SUMMARY AND CONCLUSIONS.

1) Engineering geological mapping at a scale 1:10 000 and site specific mapping at scales of 1:1500 (the Stanton Crescent Slide) and 1:1000 (the Australasian Hotel Slide), aerial photographic interpretation and laboratory testing were used to investigate landsliding in the Greymouth area, with the objectives of compiling landslide hazard zonation and land-use suitability maps suitable for urban planning and administration.

2) Landslide types identified by aerial photographic interpretation and engineering geological mapping in the Greymouth area may be grouped under the Varnes (1978) classification as: translational rock block slides and rock slides (Stanton Crescent Slide); translational debris or earth slides (Australasian Hotel Slide); rotational debris or earth slumps; flows; falls and rock avalanches.

3) The presence of steep topography (20°+) is obviously the prerequisite for landsliding within the Greymouth area. It was concluded that the bedrock/soil interface is an important discontinuity along which landsliding takes place in surficial materials and that bedding is the important discontinuity within Stillwater Mudstone along which bedrock failures take place. Landslides occurring in Cobden Limestone were typically defect controlled, by both bedding and a combination of regular spaced and highly persistent joints.

4) A detailed investigation of the Stanton Crescent Slide determined that large translational bedrock failures of this type have formed in conjunction with a marine cliffing episode *c.* 4000-5000 years B.P. There was no sign of recent activity of this slide although the potential for future movements does exist.

5) Other landslide types (rock fall, rock block slide and rock avalanche) within the field area were presumed to be caused by earthquakes. There is further work required to quantify the hazard presented by earthquake-generated landslides to both existing and future development.

6) Intense or prolonged rainfall is the primary initiating factor in the development of translational and rotational slides, and flows involving surficial materials. A record of past failures to have affected Greymouth (compiled as part of this study) indicates that landslides occur under rainstorms of differing intensity, duration and frequency. The conclusion was drawn that there is an important relationship between the antecedent soil moisture conditions and the frequency, intensity and duration of the rainstorms that cause

landsliding. Additional work is required in this area to quantify soil moisture values above which landslides may occur under differing rainfall intensities and durations.

6) Based on field investigation, it was concluded that although rainfall is the initiating agent in most failures involving surficial materials, the presence of ground water springs is the single most important factor in the development of landslides along the coastal escarpment. These springs contribute to the surcharge on the slope and reduce the cohesion of the soil. There is a clear need to investigate the hydrogeological conditions in this area. This would assist in quantifying the effects that the springs have on soil stability.

7) All soils and colluvial samples plotted under the "A" line on a Casgrande diagram classifying them as silts of very high to extra high plasticity. These results were confirmed by particle size analysis which determined that silt forms the dominant grain size with the remaining fraction formed from clay and a minor sand component. Geotechnical testing established that soil moisture increases with sample depth to the failure surface. It was also determined that in general, soil moisture values were close to the liquid limit of the associated sample. It was concluded that under intense or prolonged rainfall, the soil moisture content probably exceeds the liquid limit of the soil, thereby reducing the shear strength of the soil and contributing to landsliding. Laboratory testing identified the presence of swelling clays (Swelling Chlorite, Na Smectite and Imogolite) and these were thought to contribute to the frequency with which soils fail in the field.

8) The uncontrolled discharge of storm water over terrace edges has a major destabilising influence on the slope. Therefore, control of storm water is of paramount importance if stability is to be maintained on the slope faces underlying the terrace surfaces.

9) Landslide hazard zonation maps at a scale of 1:10 000 were compiled from the engineering geological data base. The hazard class for individual landslides was determined from the age of the most recent activity which was calculated from aerial photographs (i.e. high hazard <5 years, medium hazard 5-50 years and low hazard >50 years).

10) The hazard maps were used to develop land-use capability maps at a scale of 1:10 000. These outline the most suitable areas for future urban development and indicate the level of additional site investigation that may be required in each case. It is anticipated that the Greymouth District Council will use these maps for future urban planning and administration within the area.

11) The areas identified as most suitable for future development include the coastal plain, river flats and the terrace surfaces. Development is not recommended on the steep hill

country underlain by Tertiary limestones owing to the perceived high landslide hazard and should only proceed following detailed site investigation. Slopes underlain by Stillwater Mudstone should also only be developed following site specific investigations.

12) In areas perceived to contain some degree of hazard, the author has recommended that the Greymouth District Council require an engineering geological map accompanying subdivision proposals for the purpose of ensuring building stability. The map is to be completed by a person qualified in engineering geological or other related profession. A zone free from development is recommended along the slope toes for the purpose of protecting development from the "run out" of landslides. Additional work is required to quantify the speed and the size of the landslides that may be expected, prior to determining the width of the zone.

13) It is considered by the author that this study has fulfilled the specific objectives of the thesis (as outlined in Chapter 1) and will assist The West Coast Regional Council, and the Greymouth District Council in meeting their statutory requirements under the Resource Management Act 1991 and the Building Act 1991.

REFERENCE LIST.

Adams, J.E., 1978. Late Cenozoic erosion in New Zealand. Unpublished Ph.D. thesis, Victoria University of Wellington. 109p. + appendices.

Adams, J.E., 1980. Paleoseismicity of the Alpine Fault seismic gap, New Zealand. *Geology* Vol. 8. p72-76.

Beanland, S., and Anderson, H., 1992. The 1968 May 23 Inangahua Earthquake: An Intergrated Geological, Geodetic And Seismological Source Model. in: Nobes, D.C. (Editor) Geological Society Of New Zealand and New Zealand Geophysical Conference, 1992 Joint Annual Conference, Programme and Abstracts. *Geol. Soc. of N.Z. Misc. Pub. 63A.*

Beavis, F.C., 1985. *"Engineering Geology"* Blackwell Scientific Publications, Australia. 231p.

Bell, D.H., 1979. Engineering geology. *New Zealand Geomechanics News.* Vol 19 p20-31.

Bell, D.H., 1987. The Role Of The Engineering Geologist In Urban Planning. *Geol. Soc. Hong Kong Bull.* No. 3, pp.43-65.

Bell, D.H., 1990. Landslide Hazards in "Analysis of Natural Hazards in the Canterbury Region", Bell, D.H. ed, *CRC Report R90/8* p90-144.

Bell, D.H., 1992. in Sutherland, R.D., Kirk, R.M., and Bell, D.H. Natural Processes and Environmential Hazards In The Marlborough Sounds: Issues and Options. *An unpublished report to the Marlborough District Council.*

Bell, D.H., and Owens, I.F., 1979. High Intensity Rainstorms and Mass Movements. *Paper to ANZAAS Conference, Auckland*: p93-105.

Bell, D.H., and Pettinga, J.R., 1983. Presentation Of Geological Data. Proceedings of the Symposium: *Engineering for Dams and Canals.* I.P.E.N.Z. Vol. 9 Issue 4G.

Bell, D.H., and Pettinga, J.R., 1984. Urban Development Practices in new Zealand. Unpublished Review Paper Presented to Landplan II Workshop Held at the University of malaya, 2-3 April, 1984. 48pp.

Bell, D.H., and Pettinga, J.R., 1985. Engineering geology and subdivision planning in New Zealand. *Engng. Geol.* Vol 22. p.45-59.

Bell, D.H., and Pettinga, J.R., 1988. Bedding-controlled landslides in New Zealand soft rock terrain., in Bonnard, C. (editor) *Proc. Fifth Int. Symp. Landslides*, Vol 2, Lausanne: p77-83.

Bell, F.G., 1983. *Engineering Properties of Soils and Rocks* (second edition) Butterworths London.149.

Benn, J.L., 1992. A Review Of Earthquakes Hazards On The West Coast. Publication by The West Coast Regional Council, Greymouth. 62p.

Berrill, J.B., Bienvenu, V.C., and Callaghan, M.W., 1988. Liquefaction In The Buller Region In The 1929 And 1968 Earthquakes. *Bull. of the N.Z. Soc. for Earthquake Eng.* Vol. 21 p174-189.

Bishop, D.J., 1992. Extensional tectonism and magmatism during the middle Cretaceous to Paleocene, North Westland, New Zealand. *N.Z. J. of Geol. and Geophy.* Vol 35 (1) p81-91.

Blakeley, J.P., 1974. Determination Of Relevent Information For The Assessment Of The Stability Of Soil Slopes.*The N.Z. Geomechanics Society,* Proceedings of the Symposium on

"The Stability of Slopes in Natural Ground". N.Z.I.E. Proceedings of Technical Groups Vol. 1 Issue 5G. p6.1-6.11.

Blong, R.J., and Eyles, G.O., 1989. Landslides extent and economic significance in Australia, New Zealand and Papua New Guinea. in: Brabb, E.E., and Harrod, B.L., (Editors) *Landslides: Extent and Economic Significance*. Proc. of the 28th Int. Conf.: Symp. on Landslides. Balkema, Rotterdam.

Bowen, F.E., 1967. Early Pleistocene Glacial And Associated Deposits Of The South Island, New Zealand. *N.Z. J. of Geol. and Geophy.* 10 p164-181.

Bowen, F.E., 1975. SHEET 15 - BULLER. Geological map of New Zealand 1:250 000 NZGS.

Brabb, E.E., 1984. Innovative approachs to landslide hazard mapping. *4th Int. Symp. on Landslides*, Toronto, Vol. 1 p307-323.

Brabb, E.E., Pameyan, E.H., Bonilla, M.G., 1972. Landslide Suseptibility in San mateo County, California. *U.S. Geol. Sur. Misc. Field Map*, MF-360. (reprinted 1978).

Bradshaw, J.D., and Laird, M.G., 1980. Triassic-Jurassic rocks and Cretaceous "break-up"; sequences in the northern South Island. *Field trip guidebook for Fifth International Gondwana Symposium, Wellington.* 53p.

Bradshaw, J.D., Adams, C.J., and Andrews, P.B., 1981. Carboniferous to Cretaceous on the Pacfic margin of Gondwana: The Rangitata Phase of New Zealand. In: Creswell, M.M. and Vella, P. (Editors). Gondwana Five. Balkema. Rotterdam. pp. 217-221.

Brand, E.W., 1988. Special Lecture: Landslide risk assessment in Hong Kong. in Bonnard, C. (editor) *Proc. Fifth Int. Symp. Landslides*, Vol 2, Lausanne: p1059-1074.

Brindly, G.W., and Brown, G., (editors) 1980. *Crystal Structure of Clay Minerals and Their X-Ray Diffraction.* Mineralogical Society Monograph No. 5. London. 495p.

British Standard Association., (1979). BS 4778: Glossary of Terms used in Quality Assurance. (including relability and maintainability terms).

Broch, E., and Franklin, J.A., 1972. The point load strength test. *International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts* 9. p669-697.

Bromhead, E.N., 1986. "*The Stability of Slopes.*" Surrey University Press, New York. 373p.

Brown, E.T., (ed.) 1981. *Rock Characterization Testing and Mointoring.* ISRM Suggested Methods. Pergamon Press England. 211p.

Brown, I.R., 1974. The stability of Slopes in Soft Rocks. Report No. 112, University of Auckland, for The National Roads Board (N.Z.) 71p.

Building Act 1991. Wellington, New Zealand.

Bullock, S.J., 1978. Seismic Refraction for Site Investigation. *Bull. Ass. Eng. Geol.* Vol. 15 p19-37.

Caine, N., 1980. The rainfall intensity - duration control of shallow landslides and debris flows. Geografiska Annalar 62A (1-2) p.23-27.

Capecchi, F., and Focardi, P., 1988. Rainfall and Landslides: Research into critical precipitation coeffecient in an area of Italy. in: Bonnard, C. (Editor) *Proc. Fifth Int. Symp. Landslides* Vol. 2 p.1131-1136.

Chassie, R.G., and Goughnour, R.D., 1976. National Landslide Highway Experience. *Highway Focus*, Vol. 8, No. 1, pp.1-9.

Clayton, C.R.I., Simons, N.E., and Matthews, M.C., 1982. *Site Investigation.* Granada, London. 424p.

Cockayne, L., 1967. "*New Zealand Plants And Their Story*" (4th edition) revised E.H. Godley (editor) Govonment Printer Wellington.

Cooper, R.A., 1979. Lower Palaeozoic rocks of New Zealand. *J. of the Roy. Soc. N.Z.* 9(1), 29-84.

Cooper, A.F., and Bishop, D.G., 1979. Uplift Rates and High Level Marine Platforms Associated with the Alpine Fault at Okuru River, South Westland. in: Walcott, R.I. and Cresswell, M.M., (Editors). The Origin Of the Southern Alps. *The Roy. Soc. of N.Z. Bull.* 18 p35-43.

Costa, J.E., and Baker, V.R., 1981. Surfical Geology - "Building within the Earth", J. Wiley and Sons, First ed. 585p.

Coulter, J.D., and Hessell, J.W.D., 1980. The Frequency Of High Intensity Rainfalls in New Zealand, Part 2, Point Estimates. N.Z. Met. S. Misc. Pub. 162.

Cowan, H.A., and Pettinga, J.R., 1990. Seismic Hazards in the Canturbury Civil Defence Region. in: Analysis of Natural Hazards in the Canturbury Civil Defence Region. Canturbury Regional Council, Christchurch. p.115-142.

Crozier, M.J., 1981. Landslides in the urban environment. *Proceedings of the Symposium "Geomechanics in Urban Planning".* Institute of Professional Engineers N.Z. Proceedings of technical groups, Vol. 9 Issue 2 (G) 231-239.

Crozier, M.J., and Eyles, R.J., 1980. Assessing the Probability of mass Movement. Third Australia - New Zealand Conference on Geomechanics. IPENZ procedings of technical groups Vol. 6 Issue 2(G).

Dibble, R.R. and Suggate, R.P., 1956. Gravity Survey For Possible Oil Structures, Kotuku-Ahaura District, North Westland. *N.Z. J. of Sci. and Tech.* Vol 37 (5) p571-586.

Duncan, N., 1969. *Engineering Geology and Rock Mechanics* (Vols 1 and 2). Leonard Hill, London. 252p., 270p.

Einstein, H.H., 1988. Landslide Risk Assessment Proceedure. *Proc. of the Fifth Int. Symp. on Landslides.* Laussane p1075-1090.

Esso Exploration and Production N.Z. Ltd. 1969. Final report of marine seismic survey PPL711 (Offshore Karamea). Unpublished Petroleum Report P.R. 401 Lodged in the Library, N.Z. Geological Survey, Lower Hutt.

Eyles, R.J., and Eyles, G.O., 1982. Recognition of storm damage events. *Proc. 11th New Zealand Geog. Conf.* Wellington 1981. p.118-123.

Fairless, G.J., and Berrill, J.B., 1984. Liqefaction During Historical Earthquakes In New Zealand. *Bull. of the N.Z. Soc. for Earthquake Eng.* Vol. 17 p280-291.

Finlayson, A.A., 1984. Land surface evaluation for engineering practice: applications of the Australian PUCE system for terrain analysis. *Q. J. Eng. Geol. London*, 15, 265-316.

Flageollet, J.C., 1989. Landslides in France: A risk reduced by legal provisions. in: Brabb, and Harrod (editors) *Landslides: Extent and Economic Significance.* Balkema, Rotterdam.

Gage, M., 1952. The Greymouth Coalfield. *N.Z. Geol. Sur. Bull. 45.*

Gamble, J.C., 1971. Durability-plasticity classification of shales and other argillaceous rocks. Ph. D. Thesis, Univ. Illinois, Urbana, Illinios.

Goldwater, S., 1990. Slope Failure In Loess: A Detailed Investigation Allandale, Banks Peninsula. *An unpublished M.Sc. Thesis in Engineering Geology at the University of Canterbury.* 318p.

Grant, K., and Finlayson, A.A., 1978. The Assessment And Evaluation Of Geotechnical Resources In Urban Or regional Environments. *Eng. Geol.* Vol. 12, p219-293.

Greenway, D.R., 1987. Vegetation and slope stability in: Anderson, M.G., and Richards, K.S., (Editors). *Slope Stability*, Wiley, p.187-230.

Grey County Council List of Ordinances.

Greymouth Borough District Scheme 1987.

Grey River Argus (Newspaper) - various issues.

Greymouth Evening Star (Newspaper) - various issues.

Grim, R.E., 1962. *Applied Clay Mineralogy*, McGraw-Hill, New York.

Haast, J., 1861. Report of a topographical and geological exploration of the western districts of the Nelson province, New Zealand. Elliot, Nelson, 150 p.

Haematite Petroleum (N.Z.) Ltd. 1970. Marine Seismic Survey P.P.L. 771. (Karamea Area). *Unpublished petroleum report* P.R. 555, lodged in Geol. Sur. Lib, Lower Hutt.

Hancox, G.T., 1974. Geological Aspects Of Slope Stability.*The N.Z. Geomechanics Society,* Proceedings of the Symposium on "The Stability of Slopes in Natural Ground". N.Z.I.E. Proceedings of Technical Groups Vol. 1 Issue 5G. p4.1-4.13.

Hartlen, J., and Viberg, L., 1988. General Report: *Evaluation of landslide hazard.* Proc. Fifth Int. Symp. Landslides. Vol. 2. p1037-1059.

Hawkins, L.V., 1961. The reciprocal method of routine shallow seismic refraction investigations. *Geophysics* Vol. 26, No. 6, p806-819.

Henderson, J., 1917. The geology and mineral resources of the Reefton Subdivision, Westport and North Westland Subdivisions. *N.Z. Geol. Sur. Bull. 18.*

Hoek, E., and Bray, J.W., 1981. Rock Slope Engineering (Edition 3rd). The Institution of Mining and Metallurgy, London. 358p.

Horrey, P.J., 1989. Engineering Geological Investigation and Hazard Assessment for the Picton, Waikawa and Shakespeare Bay Area, Marlborugh Sounds. *An Unpublished M.Sc. Thesis in Engineering Geology.*

Hutchinson, G.K. and Mckie, D.A., 1979. Urban Land Use Capability Survey of Greymouth and Environs. Westland Catchment Board and Regional Water Board.

Hutchison, C.S., 1974. *Laboratory Handbook of Petrographic Techniques.* John Wiley and Sons, New York. 527p.

Hutchison, J.N., 1988. General report: Morphological and geotechnical parameters of landslides in relation to geology and hydrogeology. in *Proceedings Of The Fifth International Symposium On Landslides.* Bonnard C. (editor). Vol 1. p3-35.

Hutchinson, J.N., 1992. Landslide Hazard Assessment. *Unpublished report in Vol. 3, Proceedings of the Fifth International Symposium on Landslides.* Bell D.H. (editor).

International Society for Rock Mechanics 1984. Suggested Methods for Determining the Point Load Strength Index. ISRM committee on Field Tests, Document No. 1, pp8-12.

IPENZ 1983. Engineering Risk. Report of the Presidents Task Committee on Professional Practise and Risk. 95p.

Jessen, M.R., 1987. Urban Land Use Capibility Survey Handbook. *Water and Soil Miscellaneous Publication* No. 105. p75.

Jumikis, A.R., 1983. *Rock Mechanics* (2nd ed.) Trans Tech Publications. 613p.

Keefer, D.J., 1984. Rock avalanchs caused by earthquake - source characteristics. *Science.* Vol. 223., p.1288-90.

Koukis, G. and Ziourkas, C., 1991. Slope instability phenomena in Greece: A statistical analysis. *Int. Assoc. Eng. Geol. Bull.* Vol. 43 p47-60.

Laird, M.G., 1968. The Paparoa Tectonic Zone. *N.Z. J. Geol. and Geophys.* 11(2), p. 435-54.

Laird, M.G., 1972. Sedimentology of the Greenland Group in the Paparoa Range, West Coast, South Island. *N.Z. J. of Geol. and Geophy.* 15 P435-454.

Laird, M.G., 1988. Sheet S37 Punakaiki Geological Map of N.Z. 1:63,360 Map (one sheet) and notes (48p). Wellington, N.Z. Dept. of Scientific and Industrial Research.

Laird, M.G., 1993. Geological Aspects of the Opening of the Tasman Ocean in: Evolution of the Tasman Ocean. Van der Lingen, Swanson and Muir (Editors). *In press.* Balkema, Rotterdam.

Laird, M.G. and Shelley, D., 1974. Sedimentation and early history tectonic history of the Greenland Group, Reefton, New Zealand. *N.Z. J. of Geol. and Geophy. 17* p839-54.

Lama, R.D., and Vutukuri, V.S., 1978. *Handbook on Mechanical Properties of Rocks* Vol 4.Trans Tech Publications 515p.

Lawerence, J.H., 1981. Urban Capability Surveys As Part Of Land Use Planning. Proceedings of the Symposium "Geomechanics In Urban Planning" *IPENZ, proceedings of technical groups* Vol 9, Issue 4G.

Lewis, D.W., 1984. *Practical Sedimentology.* Hutchison and Ross, Stroudsbury, Pa. 229p.

Lewis, D.W., Shelley, D. and Bradshaw, J.D., 1980. Summary Geological History of "The Coast". *Unpublished field trip guide of the West Coast.*

Local Government Act 1974.

McKay, A., 1877. Reports relative to collections of fossils made on the West Coast district, South Island. *N.Z. Geol. Surv. Rep. of Geol. Explor. for 1882* No. 15 p91-151.

McLean, J.D., 1987. SH7 Greymouth. Omoto Slip Investigations. Engineering Geology. Unpublished Engineering Geology Immediate Report 87/001 for Works Services, Wellington.

Meneroud, J.P., Calvino, A., 1976. *Carte ZERMOS. Zones exposées à des risques liés aux mouvements du sol et du sous-sol à 1:25,000, région de la moyenne Vesubie (Alpes-Maritimes).* Orléans, Bureau de Recherches Géologiques et Minières. 11 pp., 1 map.

Mew, G., 1980. Soils of Greymouth-Hokitika Region, South Island, New Zealand and, Soil Mapping Unit Descriptions For Greymouth- Hokitika Region, South Island, New Zealand. Seperate parts of: *N.Z. Soil Survey Report 58.* New Zealand Soil Bureau, Dept. of Scientific and Industrial Research, Wellington, N.Z.

Mitchell, J.K., 1976. *Fundamentals of Soil Behavior.* John Wiley and Sons, Inc. New York. 422p.

Montgomery, C.W., 1986. "*Environmental Geology*" Wm C Brown, 1st Edition., 454pp.
Morgen, P.G., 1911. The Geology of the Greymouth Subdivision, north Westland. *N.Z. Geol. Sur. Bull. 13.*

Morgenstern, N.R., and Sangrey, D.A., 1978. Methods of Stability Analysis. Chapter 7 in "Landslides - Analysis and Control", Schuster, R.L. and Krizek, R.J. eds. *TRB Spec. Pub. 176*. p11-33.

Mostyn, G.R., and Small, J.C., 1987. Methods of stability analysis, Chapter 3 in *Soil Slope Instability and Stabilisation.* Walker, B. and Fell, R. (editors) Proceedings of an Extension Course on Soil Slope Instability and Stabilisation. A.A. Balkema. 440p.

Nathan, S., 1974. Stratigraphic nomenclature for the Cretaceous-Lower Quateranry rocks of the Bullar and North Westland, West Coast, South Island New Zealand. *N.Z. J. Geol. and Geophy.* 17(2) p423-45.

Nathan, S., 1975. Sheets S23 and S30, Foulwind and Charleston (1st ed.). Geological Map of New Zealand 1:63,360. N.Z. D.S.I.R. Wellington.

Nathan, S., 1978. Sheet S44, Greymouth (1st Edition) Geological Map of N.Z. 1:63360. Dept. of Scientific and Industrial Research. Wellington, N.Z.

Nathan, S., Anderson, H.J., Cook, R.A., Herzer, R.H., Hoskins, R.H., Raine, J.I., and Smale, D., 1986. Cretaceous and Cenozoic sedimentary basins of the West Coast Region, South Island, New Zealand. *N.Z. Geol. Survey Basin studies 1.*

New Zealand Meteorological Service, 1987. "Rainfall Observations for 1986, Stations in New Zealand, Outlying Islands." *N.Z. Met. S. Misc. Pub 110.*

New Zealand Soil Standard 4402. Parts 2.1, 2.2, 2.3.

Nilsen, T.H., Wright, R.H., Vlasic, T.C., and Spangle, W.E., 1979. Relative Slope Stability And San Francisco Bay Region, California. *U.S. Geol. Sur. Prof. Paper 944.* 96p.

Nilsen, T.H., and Turner, B.L., 1975. Influence of rainfall and ancient landslide deposits on recent landslides (1950-71) in urban areas of Contra Costa Country, California. *USGS Bulletin,* 1388, 1-18.

Northey, R.D., Hawley, J. G., Barker, P.R., 1974. Classification and mechanisms of slope failures in natural ground. *N.Z. Inst. Engr. Proc. Tech. Groups* Vol 1 issue 5(G). p31-38.

O'Loughlin, C.L., 1974. A study of tree root strength deterioration following clear felling. *Canadian Journal of Forestry Research.* Vol. 4, p.107-113.

Paterson, B.R., 1989. SH7 Omoto Slip, Mckendrys Corner, Greymouth Engineering Geological Investigations. Unpublished Engineering Geological Immediate report EGI89/042 for Works Project Services, Wellington.

Paradini, L., Guidicini, G., Bottura, J.A., Poncano, W.L., and Santos, A.R., 1977. Behaviour of the vegetation in slope stability: a citical review. *Bul. Ass. Eng. Geol.* Vol. 16, p.51-55.

Piteau, D.R., 1971. Geological factors significant to the stability of slopes cut in rock. Planning Open Pit Mines. Balkema, Amsterdam pp.33-53.

Read, S.A.L., 1974. Engineering Geological Assessment and Slope Stability. *The N.Z. Geomechanics Society,* Proceedings of the Symposium on "The Stability of Slopes in Natural Ground". N.Z.I.E. Proceedings of Technical Groups Vol. 1 Issue 5G. p4.1-4.36.

Resource Management Act 1991. Wellington, New Zealand.

Riddolls, P.M., 1987. "New Zealand Geology: A geological map of New Zealand 1:2 000 000." Department of Scientific and Industrial Research, Wellington. 70p.
Salt, G.A., 1984. Omoto Slip. Preliminary comments on geotechnical aspects of active failures affecting the road and railway.

Sassa, K., 1985. The geotechnical classification of landslides. *Proc. 5th Int. Conference and field workshop on Lanslides*, Tokyo. p.31-40.

Schuster, R.L., 1978. in: Schuster, R.L., and Krizek, R.L., 1978. (Editors) *Landslides: Analysis and Control.* Transportation Research Board Special Report 176. 234p.

Schuster, R.L., and Krizek, R.L., 1978. (editors) *Landslides: Analysis and Control.* Transportation Research Board Special Report 176. 234p.

Scott, C.R., 1980. *An Introduction to Soil Mechanics and Foundations* (3rd edition). Applied Science Publishers, London. 406p.

Selby, M.J., 1985. "*Earths Changing Surface*" Oxford, 1st Ed., 607p.

Selby, M.J., 1976. Aspects of the geomorphology of the greywacke ranges of the lower and middle Waikato Basins. J. Earth Sciences. Vol. 1, p.37-58.

Sidle, R.C., Pearce, A.J., and O'Loughlin, C.l., 1985. Hillslope stability and land use. *American Geophy. Union, Water Resources Monograph* 11.

Skempton, A.W., 1953. The collodial activity of clays, *Proc. 3rd Int. Conk. Soil Mech. Found. Engng., Zurich.* Vol 1, p57-61.

Skempton, A.W., and Hutchinson, J.N., 1969. Stability of Natural Slopes and embankment foundations. State-of-the-art Volume, *7th Int. Conference on Soil Mechanics and Foundation Engineering,* Vol. 2. p.291-340.

Smith, W.D., 1990. Earthquake Hazard In New Zealand: Some Implications of the Edgecumbe Earthquake, March 1987. *Bull. of the N.Z. Nat. Soc. for Earthquake Engng.* Vol. 23, p.211-219.

Smith, W.D., and Berryman, K.R., 1983. Revised estimates of Earthquake Hazard in New Zealand. *Bull. of the N.Z. Nat. Soc. for Earthquake Engng.* Vol. 16 p.258-272.

Sowers, G.B., and Sowers G.F., 1970. Introductory Soil Mechanics and Foundations. Macmillan, New York, 3rd edition, 556p.

Suggate, R.P., 1965. Late Pleistocene geology of the northern part of the South Island, New Zealand. *Bull. N.Z. Geol. Surv.* 77.

Suggate, R.P., 1968. The Paringa Formation, Westland, New Zealand. *N.Z. J. of Geol. and Geophy.* 13 p742-50.

Suggate, R.P., 1978. in *The Geology of New Zealand.* Suggate, R.P., Stevens, G.R., and Te Punga, M.T., (Editors). Department of Scientific and Industrial Research, Wellington. Vol. 1 and 2, 820p.

Suggate, R.P., 1985. The Glacial/Interglacial Sequences of North Westland, New Zealand. *N.Z. Geol. Survey Record 7.* 22p.

Suggate, R.P., 1987. Active folding in North Westland, New Zealand. *N.Z. J. Geol Geolphy. 30.* p169-174.

Suggate, R.P., 1988. Quaternary deposition and deformation in the Buller and tributary valleys. *N.Z. Geol Sur Record, 25.* 51p.

Suggate, R.P., and Moar, N.T., 1970. Revision of the chronology of the Late Otira glacial. N.Z. J. Geol. and Geophy. Vol. 13 p742-746.

Tulloch, A.J., 1983. Granitoid rocks of New Zealand - a brief review. *Geol. Soc. of Am. memoir 159.* p5-20.

Tulloch, A.J. and Brathwaite, R.L., 1986. Granitoid Rocks And Associated Mineralisation Of Westland-West Nelson, New Zealand. Chapter 7 in; Houghton, B.F. and Weaver, S.D. (Eds) .South Island Rocks, Tour Guides A3, C2, and C7. *New Zealand Geological Survey Record 13.* 92p.

Walcott, R.I., 1979. New Zealand Earthquakes and Plate Tectonic Theory. *Bull. of the N.Z. Nat. Soc. for Earthquake Engng.* Vol. 12, p.5-12.

Walker, B., and Fell, R., 1987 (Editors). *Soil Slope Instability and Stabilisation.* Proceedings of an Extension Course on Soil Slope Instability and Stabilisation. A.A. Balkema. 440p.

Wellman, H.W., amd Willett, R.W., 1942. The Geology of the West Coast from Abut Head to Milford Sound. *Transactions of Royal Society of N.Z.* Vol 71 p282-306.

Wellman, H.W., 1950. Tertiary and Cretaceous Geology of Sheet S44 (Greymouth). Unpublished manuscript held by the N.Z. Geol. Sur. Library, Lower Hutt.

Wellman, H.W., 1979. An uplift map for the South Island of New Zealand, a model for uplift of the Southern Alpes in: Walcott, R.I. and Cresswell, M.M., (Editors). The Origin Of the Southern Alps. *The Roy. Soc. of N.Z. Bull.* 18. p13-20.

Works Consultancy Services Geotechnical Report for Transit New Zealand. "SH 7: OMOTO SLIP McKENDRYS CORNER. 1991.

Varnes, D.J., 1978. Slope movement types and processes Chapter 2 in "Landslides - Analysis and Control", Schuster, R.L. and Krizek, R.J. eds. *TRB Spec. Pub. 176.* p11-33.

Varnes, D.J., 1984. *Landslide hazard zonation: a review of principals and practice.* Paris: UNESCO. 63 p.

APPENDIX 1.

A1.1 PLANT SPECIES FOUND IN THE STUDY AREA.

Aristotelia serrata	wineberry
Asplenium bulbiferum	hen and chicken fern
Asplenium flaccidum	drooping spleenwort
Astelia fragrans	bushlily
Astelia solandri	
Blechnum discolor	piu piu, crown fern
Blechnum chambersii	
Blechnum fluviatile	
Blechnum sp. "capense"	kio kio
Brachyglottis repanda	rangiora
Carpodetus serratus	putaputaweta
Clematis paniculata	puawhananga
Coprosma australis	kanono
Coprosma foetidissima	hupiro or stinkwood
Coprosma robusta	karamu
Coprosma rotundifolia	
Cordyline australis	cabbage tree
Coriaria arborea	tree tutu
Cortaderia richardii	toe toe
Cyathea medullaris	mamaku
Cyathea smithii	whe or soft tree fern
Cytisis scoparius	broom
Dacrydium cupressinum	rimu
Dendrobium cunninghamii	
Dicksonia squarrosa	wheki or slender tree fern
Earina autumnalis	raupeka
Earina mucronata	peka a weka or Easter orchid
Freycinatia baueriana subspecies bankskii	kiekie
Fuchsia excorticata	tree fuschia,kotukutuku
Hebe salicifolia	koromiko
Hedycarya arborea	pigeon wood
Hymenophyllum demissum	filmy fern
Hymenophyllum dilatatum	filmy fern
Hymenophyllum ferrugineum	filmy fern
Hymenophyllum flabellatum	filmy fern
Hymenophyllum rarum	filmy fern
Hymenophyllum sanguinolentum	filmy fern
Juncus species	rushes, wiwi
Leptoptheris hymenophylloides	crepe fern
Leptospermum scoparium	manuka
Lycopodium billardieri	penulous club moss
Lycopodium volubile	waewae-koukou
Melicytus ramiflorus	mahoe, whitey wood
Metrosideros diffusa	climbing rata
Metrosideros fulgens	climbing red rata, aka
Metrosideros umbellata	southern rata
Muehlenbeckia australis	pohuehue
Myrsine australis	mapou
Nertera dichondra	
Parsonia heterophylla	kaihua, New Zealand jasmine

Phormium tenax	flax, harakeke
Phymatosorus diversifolius	kowaowao, hounds tongue fern
Phymatosorus scandens	moki moki
Pittosporum tenifolium subspecies colensoi	kohuhu
Podocarpus totara	totara
Prumnopitys ferrugineus	miro
Prumnopitys taxifolius	matai
Pseudopanax colensoi	three finger
Pseudopanax crassifolius	lancewood
Pteridium esculentum	bracken
Pteris macilenta	
Rhopalostylis sapida	nikau
Ripogonum scandens	supplejack
Rubus australis	bush lawyer, tataramoa
Rubus cissoides	bush lawyer, tataramoa
Rubus fruticosus	blackberry
Schefflera digitata	pate
Sophora microphylla	kowhai
Trichomanes venosum	
Ulex europaeus	gorse
Uncinia uncinata	hook sedge
Weinmannia racemosa	kamahi

A2.1 REGIONAL GEOLOGY.

A2.1.1 PREVIOUS WORK.

The earliest comprehensive study of the geology of the West Coast was carried out by Haast (1861). Detailed mapping by Morgen (1911) and Henderson (1917) followed and was later revised and updated by Wellman (1950) and Gage (1952). Geological investigations were centred on Cretaceous and Tertiary rocks and the principle thrust was in the search for mineral resources, primarily coal, gold and oil. More recently, Suggate (1965, 1985, 1987, 1988) provided detailed accounts of the Late Tertiary and Quaternary glaciations and associated interglacials, while Laird (1968, 1988, 1993) investigated the tectonic evolution of the region. Nathan *et al.* (1986) provide the most exhaustive and up-to-date summary yet compiled of the West Coast geology and much of the following review has been based on this source.

A2.1.2 REGIONAL STRATIGRAPHY AND HISTORY.

Greymouth is located within the structurally complex and tectonically active West Coast Geologic Region, extending from Fiordland to north west Nelson. The full stratigraphy and history is not fully understood, but includes faulted and folded Mid Cretaceous - Holocene sedimentary lithologies overlying basement rocks of predominantely Paleozoic age.

Basement comprising igneous, metamorphic and sedimentary lithologies ranging in age from Precambrian to Jurrasic are represented in the West Coast region. Three major groups are recognised, Late Precambrian-Cretaceous gneiss (Charleston Metamorphic Group), Early Paleozoic metasediments, and intrusive granites (Nathan *et al.* 1986)

Most of the West Coast geologic region is underlain by the Early Paleozoic sedimentary sequence ranging in age from Cambrian to Devonian. Subdivided into three discrete successions it is the western belt consisting of Greenland Group (Figure A1.1) which extends over most of the Westland region. The Greenland Group, a flysch type sequence was deposited by turbidity currents on submarine fans during the Ordovician (Laird 1972, Laird and Shelley 1974, Cooper 1979) and is intruded by granitic rocks (Karamea Suite) of Cretaceous age (Tullock 1983). The Karamea Suite extends from the Tasman Mountains through the Paparoa and Victoria ranges southwards along the western side of the Alpine Fault and is comprised predominantly of biotite granite (Tulloch and Brathwaite 1986).

Non marine clastic sediments forming the Pororari Group (Bullock, Bovis, Watson Formations and Hawks Crag Breccia) are the oldest of the Mid Cretaceous cover lithologies (Figure A1.1). These successions represent the period of plate tectonic rifting (known as the Rangitata 2 Orogeny) that caused the separation of Australia and New Zealand during the Mid Cretaceous (Bradshaw *et al.* 1981). This phase of crustal extension (trending N to NNE) resulted in the formation of fault bounded sedimentary basins, into which Late Cretaceous - Tertiary sediments including the Pororari Group were deposited (Bradshaw and Laird 1988, Laird 1968, 1988, 1993).

The Mawheranui Group (Late Cretaceous - Tertiary in age) unconformably overlie the Pororari Group and include the Paparoa and Brunner Coal Measures (Figure A1.1). In contrast to the Paparoa Coal Measures which consist of an alternating sequence of lacustrine and fluvial sediments, the Brunner Coal Measures are entirely fluvial in origin. Deposition of the overlying Kaiata Formation occurred in the Eocene during a marine transgression resulting from renewed crustal extension. This tectonic phase continued until the end of the Oligocene at which time the various limestones of the Nile Group were deposited. A change in the tectonic regime from oblique extension to oblique compression during the Late Oligocene signalled the end of the marine transgression. It is widely accepted that the development of a transform plate boundary, now forming the Alpine Fault, also occurred at this time (Nathan *et al.* 1986). Uplift of the submerged land mass in response to increasing compression along the plate boundary caused a wide-spread marine regression in the Miocene. Massive marine mudstones forming the Blue Bottom Group were deposited during this time.

Sedimentation during the Quarternary - recent period occurred during a series of alternating glacial and interglacials (Table A1.1) (Nathan 1978). During the main glacial advances, the upper reaches of the major river valleys were infilled with ice, and extensive aggradational terrace surfaces formed downstream from the terminal moraines of these glaciers (Nathan 1978).

Table A1.1 Nomenclature of the north westland glacial/interglacial sequence. Based on Suggate (1985).

Otira Glaciation	Kaihinu Interglacial	Waimea Glaciation	Karoro Interglacial	Waimaunga Glaciation	Scandinavia Interglacial	Nemona Glaciation	Unnamed Interglacial
Moana Formation	Awatuna Formation	Waimea Formation	Karoro Formation	Tansey Formation	Scandinavia Formation	Cockeye Formation	
Larrikins Formation	Rutherglen Formation						
Loopline Formation							

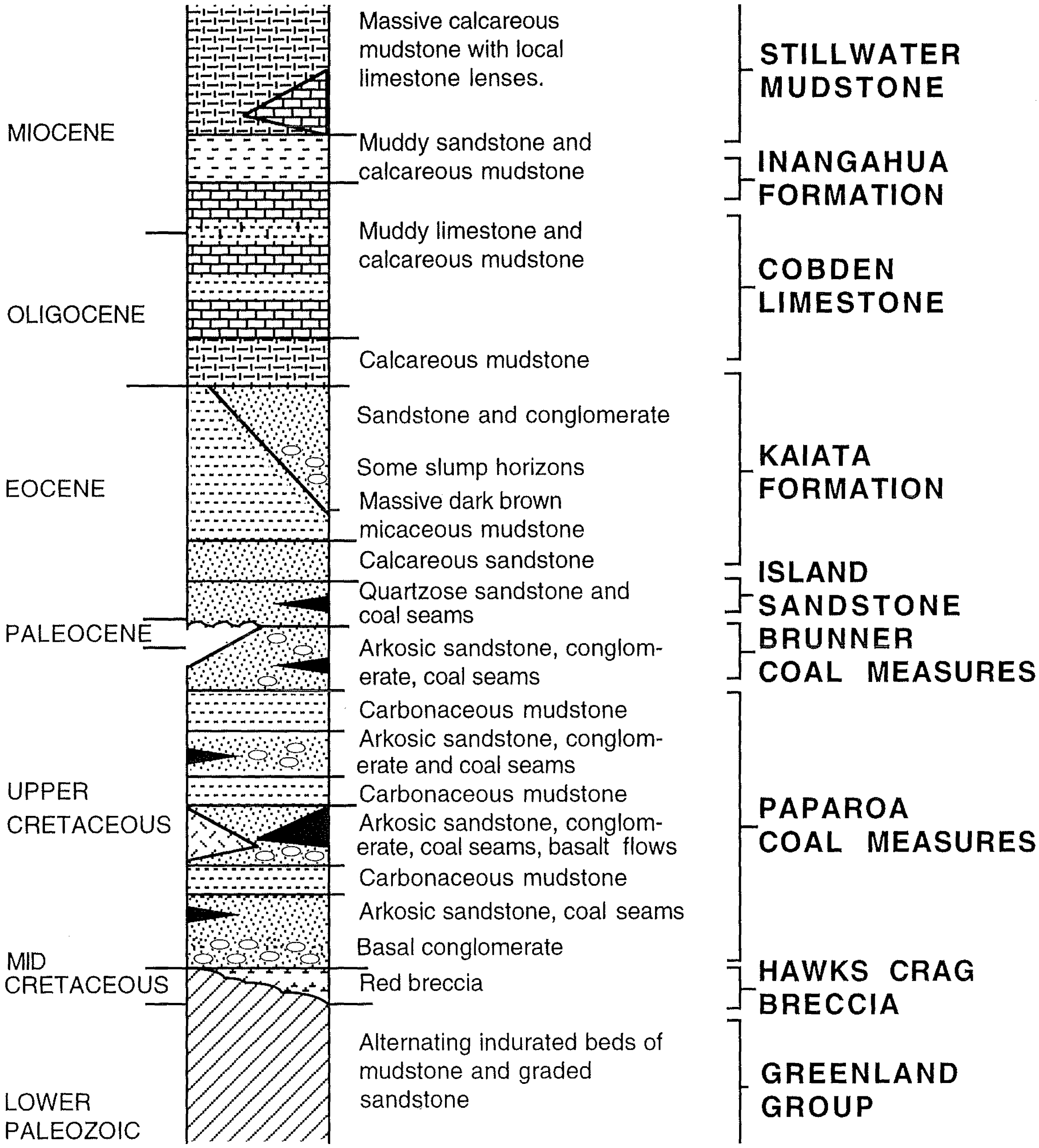


Figure A1.1. Composite stratigraphic column (not to scale) for the Cretaceous and Tertiary rocks of the Greymouth area. Redrawn from Lewis *et al.* 1980.

A1.3 GEOLOGICAL TIME SCALE

Eras of time	Periods of time	Epochs of time *(Cenozoic era only)*	Age *(millions of years)*	Duration *(millions of years)*
Cenozoic	Quaternary	Recent		2
		Pleistocene	2	
	Tertiary	Pliocene	5	23
		Miocene	25	
		Oligocene	38	40
		Eocene	55	
		Paleocene	65	
Mesozoic	Cretaceous		144	79
	Jurassic		213	69
	Triassic			35
Paleozoic	Permian		286	38
	Carboniferous		360	88
	Devonian		408	48
	Silurian		438	30
	Ordovician		505	67
	Cambrian		590	85
Precambrian			590-4000	3520

A1.4 SLOPE STABILITY - A REVIEW.

A1.4.1 TERMINOLOGY.

Landslides are included in the group of slope movements wherein shear failure occurs along a specific surface or combination of surfaces (Schuster 1978) and involve the predominantly downslope transfer of geological materials under the influence of gravity (Bell 1990). A variety of terms have been used in the literature including "landslide" (Skempton and Hutchinson 1969; Varnes 1978), "mass wasting", mass movement", "mass transport" (Selby 1985, Montgomery 1986), and "landslip" (Northey *et al.* 1974). A distinction may also be made on the basis of the definition of surface form, rate of movement, slope angle or a combination of these. Crozier (1986) recognised "landslides" as areally discrete whilst "slow and distributed movements" are ill defined extensive areas of ground creep. In Japan, the distinction is made between "landslides" which are slow movements on gentle slopes and "slope failures" which are fast movements on steep slopes (Sassa 1985).

The term "landslip" has been legally defined in New Zealand under the 1970 amendment to the Earthquake and War Damage Act 1944 as:

> *"...the subsidence of a substantial land mass other than by settlement, soil shrinkage or compaction; including the movement from any hill, mound, bank, slope,cliff or face of earth or rock of a substantial mass of earth or rock which before movement formed an ntegral part of any hill, mound, bank, slope, cliff or face".*

As Bell (1990) points out, there is considerable difficulty in defending such a definition technically as the individual terms (subsidence, settlement, compaction) used in the definition have been left undefined, and confusion arises over the use of "subsidence". The term subsidence is generally regarded as the vertical downwards movement of the ground caused by the removal of the underlying support (Costa and Baker 1981).

Varnes (1978) recognised "slope movements" as the correct technical term whilst continuing to use "landslides" with the same meaning. "Slope movements" (and landslides) thus involve "... *the essential downslope transfer of geological materials under the influence of gravity*", whilst "*ground subsidence*" *is* "... *the essentially vertical downwards movement of geological materials consequential on the removal of underlying support*" (Bell 1990).

A1.4.2 CLASSIFICATION.

"*Slope movement classification is an attempt to reduce a multitude of different but related slope processes, to a few easily recognisable and meaningful groups on the basis of common properties*" (Goldwater 1990). As a nearly continuous gradation exists in both type of movement and material, a rigid classification is neither practical or desirable (Varnes 1978, Walker *et al.* 1987). The difficulty is in developing concise, unambiguous descriptive terminology, that can be universally applied by individuals unfamiliar with the subject.

In the classification developed by Varnes (1978), a concise and widely used terminology has been compiled (Figure 1.5). In order of importance, the principle criteria that have been used in the classification are:

1) type of movement;
2) type of material;
3) landslide geometry;
4) geological morphological and climatic factors;
5) causes of movement; and
6) age and state of activity.

Six fundamental types of movement are recognised in the Varnes (1978) classification and these are: slides; flows and lateral spreads; falls and topple; complex slope movements. Failure involves either sliding, flowing, falling or a combination of these movements. "Sliding" motion involves shear displacement along one or more discrete failure surfaces; "flows" involve an internal shearing motion resembling that of viscous fluids; "falls" involve a component of free fall in their motion while "topples" are essentially a pivotal motion about a centre of rotation. In many cases it is common to discover that failure may include a combination of these movements and these slope failures are termed complex slope movements.

In the Varnes (1978) classification, "material type" has been divided into "bedrock" and "engineering soil", which has been further subdivided into "debris" (coarse grained engineering soils), and "earth" (fine grained engineering soils). Debris includes at least 20 percent of fragments finer than 2mm whilst earth contains 80 percent finer than 2mm. Description of the materials involved in failure is necessary in a classification since the nature of the failure material strongly influences the type and degree of landsliding that develops.

Varnes (1978) also provides for some description of the speed (Figure 1.5) at which the failure occurs. This is important in evaluating the hazard and risk for each type of landslide in a given area. A landslide failing at a few mm/annum will pose little hazard (assuming that the rate of movement does not increase) whereas, the same landslide failing at several 10's of m/minute would pose a very much greater threat to human life and property. Rates of movement in the Varnes (1978) classification range from extremely rapid (3.0 m/s) to extremely slow (0.06 m/year).

A1.4.3 CAUSES OF SLOPE MOVEMENTS.

The processes involved in initiating mass movements comprise a continuous series of events, from cause to effect. In reality however, it is unlikely that a landslide can be attributed to a single definite cause (Varnes 1978). As Sowers and Sowers (1970) point out:

"In most cases a number of causes exist simultaneously and so attempting to decide which one finally produced failure is not only difficult but also incorrect. Often the final action is nothing more than a trigger that set in motion an earth mass that was already on the verge of failure."

For this reason it is usual to recognise both pre-existing and initiating factors (Bell 1990). Since all landslides involve the failure of earth materials under shear stress, it follows therefore that causative factors may be grouped according to the following:

1) those factors contributing to an increase in shear stress (= driving forces); and

2) those factors that lead to low or reduced shear strength (= resisting forces) (Varnes 1978).

Factors that fall into the former category include: removal of lateral support; surcharge (for example rain or engineered fill); removal of underlying support and lateral pressure, whilst factors in the second category include: changes due to weathering and other physicochemical reactions; changes in intergranular forces due to water content and pressure in pores and fractures; and changes in structure (Varnes 1978). A comprehensive list of factors compiled by Koukis and Ziourkas (1991) that are known to contribute to landsliding is given in Table A1.2.

Table A1.2 Causes of landslide movements (after Koukis and Ziourkas 1991).

A. Factors contributing to increased shear stress.		
Transitory earth stresses	-Earthquakes -Vibrations from blasting -Vibrations from machinery -Vibrations from traffic -Adjacent slope failures	
	-Squeezing out of underlying plastic material	
Removal of lateral and underlying support	-Erosion, undercutting of banks	-Streams and Rivers -Waves -Tidal currents -Precipitation -Subaerial weathering -Solution and removal of material
	-Creation of new slopes	-Previous movements -Large scale faulting
	-Work of human agencies	-Cuts, pits etc. -Removal of retaining technical works -Creation of reservoirs
Surcharge	-Natural agencies	-Weight of precipitation -Accummulation of talus -Vegetation
	-Work of human agencies	-Fills -Stockpiles of ore -Stockpiles of rock -Stockpiles, rubbish heaps -Buildings and other structures -Weight of water from leaking pipes etc.

	-Cultivation and irrigation of slopes	
Increase of the slope angle through regional tilting-local raising		
Volcanic processes		
Lateral pressure	-Water in cracks and caverns	
	-Freezing of water in cracks	
	-Swelling of clay or anhydrite	
	-Mobilisation of residual stress	

B. Factors contributing to low or reduced shear strength.

Changes due to weathering and other physio-chemical reactions	-Softening of fissured clays	
	-Physical disintegration of granular rocks	
	-Hydration of clay minerals	
	-Drying of clays	
	-Removal of cement by solution	
	-Drying of shales	
	-Migration of water to weathering front under electric potential	
Initial state	-Composition	
	- Texture	-Fractures
		-Faults
		-Bedding planes
		-Foliation in schists
		-Cleavage
		-Brecciated zones
		-Massive beds over weak or plastic zones
		-Alteration of permeable and impermeable beds
		-Slope orientation
	-Gross structure and slope geometry	
Changes of intergranular forces due to water content and pressure in pores and fractures	-Rainfall	
	-Snowmelt	
	-Diversion of streams	
	-Ponds	
	-Reservoirs	
	-Irrigation	
	-Clearing of vegetation and deforestation	

Changes in structure
Weakening due to progressive creep
Action of tree roots
Action of burrowing animals

A1.4.3.1 Rainstorm generated failures.

Climate, in particular intense or prolonged rainfall, plays an important role in initiating shallow (<2m deep) landslides on steep slopes (20+°) where regolith materials overlie impermeable bedrock (Bell 1992). A survey by the Federal Highway Administration in the United States concluded that "water is the controlling factor or a major contributing factor in about 95 percent of all landslides" (Chassie and Goughnour 1976). The effects of water on surficial geologic materials have been summarised by Bell and Owens (1979) as:

1) magnitude-duration-frequency relationships for specific storm events;
2) antecedent regolith moisture history;
3) ground water distribution and seepages;
4) types of vegetation cover and root binding effects;
5) slope angle and aspects;
6) regolith depth, profile geology and weathering characteristics;

7) nature and orientation of potential failure surfaces; and
8) geomechanical parameters at these failure surfaces.

Eyles and Eyles (1982) investigated erosion-causing storms in New Zealand and found that rainfall intensities causing these are of two types. The first is high intensity events with rainfall ranging as high as 900mm over three days, the second is long periods of near capacity soil moisture levels as a result of prolonged, light rainfall with additional relatively small triggering storms (Blong and Eyles 1989). Storm events are assumed evenly distributed in time. Therefore it is possible to assign a probability of reoccurrence to individual magnitude storm events (i.e. 1 above). However as Eyles and Eyles (1982) noted, this can be extremely difficult owing a) to the often localised nature of storm events and b) to the very dispersed rainfall measuring network in rural areas.

The fact that 1) above is possible has been illustrated by Bell and Owens (1979) who concluded that; reoccurrence intervals for 6 and 12 hour duration rainfalls occurring during the passage of Cyclone Alison in March 1975, exceeded 100 years.

Crozier and Eyles (1980) have developed an antecedent excess rainfall model and applied this to Wellington city, using the cities rainfall events and known landslide damage events. These authors also identified threshold values for rainfall above which landsliding may occur. The importance of antecedent regolith moisture history (2 above) in initiating landslides is well known (Capecchi and Facardi 1988, Crozier and Eyles 1980, Selby 1976, Nilson and Turner 1975). Soil moisture is a function of the particle size of the constituent grains, void spaces, evaporation history (sun and wind), plant cover and past rainfall. Since the effects of past rainfall on soil moisture may be represented by an exponential decay curve, in practise it is usual to consider only that precipitation occurring within the last 30 days (Capecchi and Focardi 1988). It must be remembered that antecedent rainfall conditions are site specific and that generalising over a wide area is less likely to produce meaningful results (Caine 1980).

Vegetation plays an important role in the stability of a slope and the effects may be grouped into two broads areas:

1) hydrological factors; and
2) mechanical factors.

A list of factors included under these two categories is summarised in Table A1.3. Previous reviews on the effects of vegetation on hill slope stability include Greenway (1987), O'Loughlin (1974), Prandini *et al.* (1977), Sidle *et al.* (1985) and for greater detail the reader is referred to these.

Table A1.3 The effects of vegetation on the stability of slopes (Greenway 1987).

Hydrological Mechanisms	Mechanical Mechanisms
1. Foliage intercepts rainfall causing absorptive and evaporative losses that reduce rainfall avaliable for infiltration. (B)	5. Roots reinforce the soil, increasing soil shear strength. (B)
2. Roots and stems increase the roughness of the ground surface and the premeability of the soil, leading to increased infilitration capacity. (A)	6. Tree roots may anchor into firm strata, providing support to the upslope soil mantle through buttressing and arching (B)
3. Roots extract moisture from the soil which is lost to the atmosphere via transpiration, leading to lower pore-water pressureses. (B)	7. Weight of trees surcharges the slope, increasing normal and downhill force components. (A) (B)
4. Depletion of soil moisture may accentuate desiccation cracking in the soil, resulting in higher infiltration capacity. (A)	8. Vegetation exposed to the wind transmits dynamic forces into the slope. (A)
Legend: A-adverse to stability; B-benefical to stability	9. Roots bind soil particles at the ground surface, reducing their susceptibility to erosion. (B)

A1.4.3.2 Earthquake generated slope movements.

Landslides triggered by earthquakes may occur in at least three ways, including:

1) by horizontal and vertical accelerations (up to 0.5g), altering the distribution of forces within hillslopes in a manner analogous to the over steepening of the slope;

2) rapid and repeated stress flucations can induce changes in pore fluid pressure distribution reducing soil strength; and

3) earthquake shaking may decrease soil cohesion by breaking intergranular bonds (Walker and Fell 1987).

Earthquakes are measured in terms of "magnitude" (M) and "intensity" (MM). Earthquake magnitude is measured instrumentally and is a record of the amount of energy released during the earthquake. The intensity of an earthquake is a measure of effects felt at the ground surface and varies according to the proximity of the epicentre, local geology and engineered structures (Cowan and Pettinga 1990).

As the intensity of an earthquake decreases with distance from the epicentre, it seems logical to assume that the area affected by landslides during an earthquake will be proportional to the earthquake magnitude (Walker and Fell 1987). Keefer's (1984) analysis of 40 historical earthquakes world wide indicates that the area in which landslides occur increases from 0 square kilometres at Richter magnitude 4.0, to 500,000 square kilometres at magnitude 9.2. In addition, Keefer (1984) suggested that rock falls, rock slides, soil falls, and soil slides can be triggered by the weakest seismic activity while deep seated slumps and earthflows are generally initiated by stronger ground shaking. Lateral spreads and debris flows require the greatest seismic activity (Sidle *et al.* 1985).

A1.4.4 SLOPE STABILITY ASSESSMENT.

In an assessment of slope stability, it is usual to recognise a number of phases. Based on Walker and Fell (1987) these include:

1) Phase One: Initial studies. This involves background research in an attempt to understand the area and its problems. Methods include investigation of the regional geology, topography, climate, vegetation, aerial photographic interpretation, and planning of phase 2.

2) Phase Two: Field Investigations. The objective of these is to determine site specific conditions through field mapping, subsurface investigation, *in situ* testing and field monitoring. The methods employed and the techniques used depend on the nature of the problem and on the level of funding.

3) Phase Three: Laboratory Testing. The purpose of these is to determine engineering parameters for use in stability analysis.

4) Phase Four: Data Presentation. The objective of these is the production of geotechnical failure models and the presentation of site information.

Often the stability of slope is expressed as a "factor of safety" against sliding (Scott 1980). The factor of safety is defined as.

> "*that factor by which the shear strength parameters may be reduced in order to bring the slope into a state of equilibrium along a given slip surface*" (Morgenstern and Sangrey 1978).

A factor of safety $F \geq 1$ indicates that the slope is stable under current conditions. If $F < 1$ then failure of the slope can be expected if it has not occurred already.

The effect of pore pressures is to reduce the stability of the soil mass and therefore the factor of safety. Pore pressures are buoyancy forces that form from the presence of free water in the soil voids. The water exerts a hydrostatic stress during undrained soil conditions. Drainage of the soil mass allows pore pressures to dissipate, enhancing stability (Bromhead 1986). Cohesion results from the attractive forces between particles, in particular the clay minerals. Cohesion acts to resist shearing motion, thereby enhancing slope stability.

In a given area most of the causes of slope stability can be identified; some may be mapped and correlated to each other and with existing failures. The extension of accurate site or point information then allows the evaluation of larger susceptible areas. This is the approach that is commonly used in landslide hazard zonation.

ENGINEERING GEOLOGICAL FIELD DESCRIPTION FOR ROCK MATERIAL

WEATHERING

	TERM	GRADE	ROCK DESCRIPTION
6.	* residual soil (RW)	VI	discolouration and complete transformation to soil; original fabric destroyed
5.	completely weathered (CW)	V	discolouration and transformation to soil; original fabric largely preserved
4.	highly weathered (HW)	IV	material pervasively altered with discolouration and loss of strength; fabric preserved; lithorelicts
3.	moderately weathered (MW)	III	penetrative discolouration and alteration of rock material, with some loss of strength
2.	slightly weathered (SW)	II	slight discolouration of rock fabric; no loss of material strength
1.	unweathered (UW)	I	no discolouration or loss of strength, or any other effects due to weathering

STRENGTH

	TERM	POINT LOAD STRENGTH INDEX Is(50)	FIELD ESTIMATION OF STRENGTH
1.	extremely strong (ES)	more than 10	can only be chipped with geological hammer
2.	very strong (VS)	3 to 10	several hard blows required to break hand specimen
3.	strong (S)	1 to 3	few firm blows of hammer required to break specimen
4.	moderately strong (MS)	0·3 to 1	breaks readily with one blow of hammer
5.	moderately weak (MWk)	0·1 to 0·3	broken by hand only with difficulty; small thin pieces broken by finger pressure
6.	weak (Wk)	0·03 to 0·1	broken by hand; pieces 25 mm or more broken by finger pressure
7.	* very weak (VWk)	less than 0·03	crushed or remoulded by hand (grades into soil materials)

* may require description as soil material

GEOLOGICAL CLASSIFICATION

SEDIMENTARY (†)

CRYSTAL OR GRAIN SIZE (mm)	CLASTIC	CHEM/ORGANIC
very coarse 64; coarse 2	CONGLOMERATE (1) AGGLOMERATE (2) BRECCIA (3)	
medium	SANDSTONE (4)	Calcareous: LIMESTONE (9) Siliceous: CHERT (10), OTHER (11)
0·06 — fine	TUFFS (7) SILTSTONE (5) MUDSTONE (6)	Carbonaceous: COAL (12), OTHER (13) Ferruginous: LATERITE (14), OTHER (15)
very fine 0·002	CLAYSTONE (8)	Saline: ROCK SALT (16), GYPSITE (17), OTHER (18)

IGNEOUS (†)

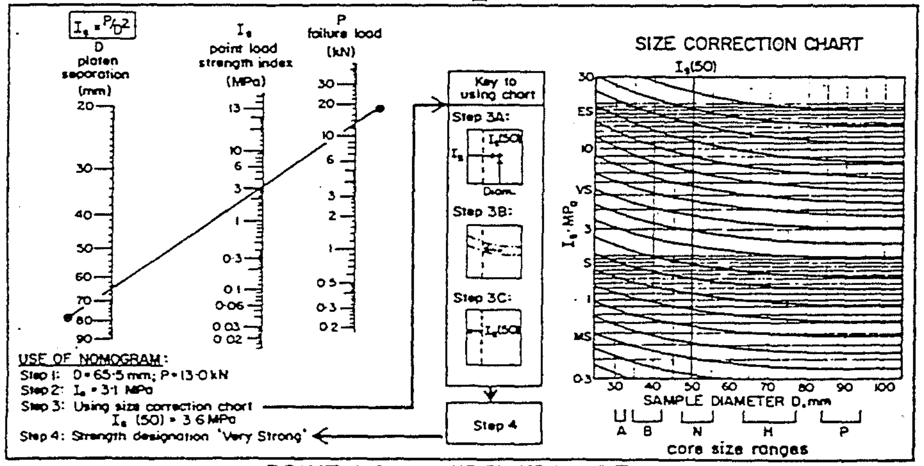
Mode of Occurrence	Silicic	Intermed	Mafic	Ultramafic
INTRUSIVE	GRANITE (27) GRANODIORITE (28)	SYENITE (29) DIORITE (30)	GABBRO (31)	PERIDOTITE (32) DUNITE (33)
EXTRUSIVE	RHYOLITE (19) OBSIDIAN (20) DACITE (21)	TRACHYTE (22) ANDESITE (23)	DOLERITE (24) BASALT (25)	SERPENTINITE (26)

METAMORPHIC (†)

FOLIATED	MASSIVE
GNEISS (34)	HORNFELS (39)
SCHIST (35)	MARBLE (40)
PHYLLITE (36)	QUARTZITE (41)
SLATE (37)	AMPHIBOLITE (42)
MYLONITE (38)	

(†) OTHERS: Specify (43)

--- WEATHERING TERM	--- STRENGTH TERM	--- --- --- COLOUR	--- FABRIC	--- --- ROCK NAME



POINT LOAD STRENGTH INDEX

1: light 2: dark	1: pinkish 2: reddish 3: yellowish 4: brownish 5: olive 6: greenish 7: bluish 8: whitish 9: greyish	1: pink 2: red 3: yellow 4: brown 5: olive 6: green 7: blue 8: white 9: grey 0: black

COLOUR

1: finely layered (<25 mm)
2: coarsely layered (25 - 100 mm)
3. massive
4. other (specify)

FABRIC

ENGINEERING GEOLOGICAL FIELD DESCRIPTION FOR SOIL MATERIAL

WEATHERING

	TERM	GRADE	SOIL DESCRIPTION
5	Completely Weathered (CW)	V	completely discoloured and altered, no trace of original fabric
4	Highly Weathered (HW)	IV	mostly altered and weakened, some trace of original fabric
3	Moderately Weathered (MW)	III	large discoloured portions of original soil separated by more altered material, significantly weakened
2	Slightly Weathered (SW)	II	minor discolouration of some parts of the original soil, no loss of strength
1	Unweathered (UW)	I	original soil with NO discolouration, loss of strength or other effects due to weathering

0 NOTE * in coarse-grained soils record weathering grade of DOMINANT fraction here and qualify weathering grade of subordinate and/or minor fractions if appropriate

STRENGTH

	TERM	FIELD CRITERIA
1	loose	can be removed from exposure in disaggregated form by hand
2	compact	only removed from exposure by implement; material readily disaggregated by physical means
3	+ cemented	only removed from exposure by implement; material does not disaggregate
4	hard	may be removed from exposure with difficulty by implement or hand, softened on immersion in water, may be remoulded
5	stiff	indented by thumb pressure, but not moulded by fingers; softened on immersion in water, and may be remoulded
6	firm	moulded or indented only by strong finger pressure, easily moulded after immersion in water
7	soft	easily indented or moulded by finger pressure
8	very soft	exudes between fingers when squeezed
9	spongy	readily compressed by finger pressure, but cannot be remoulded

+ may require description as rock material

UNIFIED SOIL CLASSIFICATION SYSTEM

FIELD IDENTIFICATION						GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS	GRAVELS (>50% larger than 2mm)	clean gravels	wide range in grain size and substantial amounts of all interm. sizes			GW	well graded GRAVELS
			predom. one size or a range of sizes with some interm. sizes missing			GP	poorly graded GRAVELS
		gravels with fines	non-plastic fines (see ML below)			GM	poorly graded SILTY-GRAVELS
			plastic fines (see CL below)			GC	poorly graded CLAYEY-GRAVELS
	SANDS (>50% smaller than 2mm)	clean sands	wide range in grain sizes and substantial amounts of all interm. sizes			SW	well graded SANDS
			predom. one size or a range of sizes with some interm. sizes missing			SP	poorly graded SANDS
		sands with fines	non-plastic fines (see ML below)			SM	poorly graded SILTY SANDS
			plastic fines (see CL below)			SC	poorly graded CLAYEY-SANDS
FINE-GRAINED SOIL	SILTS AND CLAYS	LIQUID LIMIT	SHINE	DILATANCY (1)	TOUGHNESS (1)		
		<50	none to very dull	quick to slow	none	ML	INORGANIC SILTS with slight plasticity
		<50	moderate	none to very slow	medium	CL	INORGANIC CLAYS of low to medium plasticity
		<50	none to very dull	slow	slight	OL	ORGANIC SILTS & CLAYS of low plasticity
		>50	dull	slow to none	slight to medium	MH	INORGANIC SILTS of high plasticity
		>50	very glossy	none	high	CH	INORGANIC CLAYS of high plasticity
		>50	moderate to v. glossy	none to very slow	slight to medium	OH	ORGANIC CLAYS of medium to high plasticity
HIGHLY ORGANIC SOILS			identified by colour, odour, spongy feel and fibrous texture			Pt	PEAT and other highly organic soils

PROCEDURES FOR FINE-GRAINED SOILS OR FRACTIONS (1)

DILATANCY (reaction to shaking) -

1) Prepare pat of moist soil, adding water to make soft - but not sticky

2) Place pat in palm of hand, shake horizontally by striking vigorously against other hand

POSITIVE REACTION appearance of water on surface of pat, which becomes glossy. When squeezed between fingers, water and gloss disappear, pat stiffens and may crumble

TOUGHNESS (consistency near plastic limit) -

1) Mould sample to consistency of putty, adding water or air drying as required

2) Roll to thin (3mm) thread, fold and reroll repeatedly until thread crumbles at plastic limit

3) Knead together and continue until lump crumbles

Diagnosis: a tough thread and stiff lump indicate high plasticity, a weak thread and lump low plasticity clays

GROUP SYMBOL CODINGS FOR USCS

COLUMN 1		COLUMN 2	
G 1	C 4	W 1	C 4
S 2	O 5	P 2	L 5
M 3	Pt 6	M 3	H 6

BOUNDARY CLASSIFICATIONS specify, enter 0,0

WEATHERING TERM | WATER CONTENT TERM | STRENGTH TERM | COLOUR | FABRIC | SOIL NAME | USCS SYMBOL

WATER CONTENT

	TERM	FIELD CRITERIA
1	Dry	looks and feels dry, fine-grained soils usually hard, powdery or friable, coarse-grained soils may run freely through hands
2	Moist	soil feels cool and may be darkened in colour, particles tend to adhere in coarse-grained materials, fine-grained soils may be softened
3	Wet	soils feel cold and are darkened in colour, free water forms on hands when sample is disturbed
4	Saturated	restricted to wet soils below the water table or the static water level in excavations or drill holes

COLOUR

1: light	1 pinkish	1 pink
2: dark	2 reddish	2 red
	3 yellowish	3 yellow
	4 brownish	4 brown
	5 olive	5 olive
	6 greenish	6 green
	7 bluish	7 blue
	8 greyish	8 white
		9 grey
		0 black

FABRIC

1: finely layered (25 mm)

2: coarsely layered (25-100 mm)

3: massive

4: other (specify)

SOIL NAME

*SUBORDINATE FRACTION — 20-50% volume visual estimate

DOMINANT FRACTION — >50% volume visual estimate

* MINOR FRACTION — <20% volume visual estimate

Subordinate fraction	
1 coarse	gravelly
2 medium	gravelly
3 fine	gravelly
4 coarse	sandy
5 medium	sandy
6 fine	sandy
7 silty	
8 clayey	
9 peaty	

PARTICLE SIZE

SOIL TYPE TERM		PARTICLE SIZE (mm)	GRAPHIC LOG
1 coarse	gravel	> 60	
2 medium	gravel	20-60	
3 fine	gravel	2-20	
4 coarse	sand	0.6-2.0	
5 medium	sand	0.2-0.6	
6 fine	sand	0.06-0.2	
7 silt		0.002-0.06	
8 clay		< 0.002	
9 peat		NA	

WITH SOME (minor fraction)	
1 coarse	gravel
2 medium	gravel
3 fine	gravel
4 coarse	sand
5 medium	sand
6 fine	sand
7 silt	
8 clay	
9 peat	

ENGINEERING GEOLOGICAL FIELD DESCRIPTION FOR ROCK MASS

FIELD SKETCH OR ANNOTATED PHOTOGRAPH

GENERAL INFORMATION	SHEET REFERENCE
PROJECT/AREA	
LOCALITY	
REFERENCE MAP	ADDITIONAL DATA
COORDINATES	
ELEVATION	
GEOLOGY BY	
DATE	
CHECKED	
Sheet No of	

SCALE — ORIGINAL SCALE 1 — SKETCH ORIENTATION

GRAPHIC SYMBOL	ROCK UNIT No	ROCK MATERIAL DESCRIPTION				
		WEATHERING	STRENGTH	COLOUR	FABRIC	ROCK NAME
	1					
	2					
	3					
	4					
	5					
	6					
		OTHER MATERIAL DESCRIPTIONS (specify)				
	7					
	8					
	9					

TERMINOLOGY FOR COMMON DEFECTS IN ROCK MASS

DEFECT TYPE	LAYERING (1)			FRACTURES AND FRACTURE ZONES (1)		WEAK SEAMS OR ZONES (1)		
	BEDDING (1)	FOLIATION (2)	CLEAVAGE (3)	JOINT/FAULT (4)	SHEARED ZONE (5)	CRUSHED (6)	DECOMPOSED (7)	INFILLED (8)
DESCRIPTION	i) Arrangement in layers of mineral grains or crystals of similar size or composition ii) Arrangement of elongated or platy minerals near parallel to one another and/or to the layers			Single fracture across which rock has little tensile strength, planar, curved, or irregular, open, closed or incipient, surface rough, smooth or slicken-sided	Zone of multiple closely spaced (<100 mm) fracture planes, with roughly parallel planar boundaries, blocks of intact material within zone typically lenticular or wedge shaped, fractures closed, open, weakly cemented or soil-coated	Zone with roughly parallel planar boundaries - i) Composed of disoriented, usually angular, rock fragments of variable size in a soil matrix ii) Some weathering of fragments possible, with soils either cohesive or non-cohesive iii) Lowest shear strength parallel to zone boundaries, which are commonly slickensided	Zone of any shape, but commonly with roughly parallel planar boundaries - i) Within zone rock moderately to completely weathered ii) At margins gradation into fresher rock NOTE materials forming zone require separate description as soil or weak rock	i) Composed of soil materials which may show layering roughly parallel to zone boundaries ii) Rock fabric not present in infill zone
MAP SYMBOLS with strike and dip	016 73	60 064	37 124	'open' 068 27 'closed' 015 86	45 063	78 072	20 069	064 20

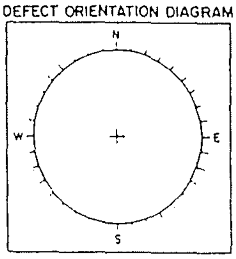
★ Record width of zone as required

(1) OTHERS Specify (9)

DEFECT DATA SUMMARY TABLE

ROCK UNIT No	DEFECT SET No	DEFECT TYPE	DEFECT SPACING	DEFECT PERSISTENCE	AV UNIT BLOCK SIZE	DEFECT ORIENTATION		GROUND-WATER	REMARKS
						DIP DIRECTION	DIP VALUE		

DEFECT ORIENTATION DIAGRAM



DEFECT SPACING

	TERM	SPACING (mm)
1	extremely wide	> 2000
2	very wide	500 – 2000
3	wide	200 – 500
4	moderate	100 – 200
5	close	25 – 100
6	very close	5 – 25
7	extremely close	< 5

DEFECT PERSISTENCE

	TERM	LENGTH (m)
1	very high	> 10
2	high	5 – 10
3	moderate	2 – 5
4	low	0.5 – 2
5	very low	< 0.5

AVERAGE UNIT BLOCK SIZE

	TERM	EQUIVALENT CUBE SIDE	APPROX BLOCK VOLUME (m^3)
1	very large		> 10
2	large	1000 mm	0.1 – 1.0
3	medium	500 mm	0.001 – 0.1
4	small	100 mm	
5	very small	10 mm	< 0.001

GROUNDWATER

	TERM	FLOW RATE
1	dry	
2	seepage	< 1 ml s^{-1}
3	very low flow	1 – 10 ml s^{-1}
4	low flow	10 – 100 ml s^{-1}
5	moderate flow	0.1 – 1 l s^{-1}
6	large flow	1 – 10 l s^{-1}
7	very large flow	> 10 l s^{-1}

ENGINEERING GEOLOGICAL FIELD DESCRIPTION FOR SOIL MASS

FIELD SKETCH OR ANNOTATED PHOTOGRAPH

SCALE ORIGINAL SCALE 1 SKETCH ORIENTATION

GENERAL INFORMATION	SHEET REFERENCE
PROJECT/AREA LOCALITY	
REFERENCE MAP COORDINATES ELEVATION	ADDITIONAL DATA
GEOLOGY BY DATE CHECKED Sheet No. of	

SOIL MATERIAL DESCRIPTION

GRAPHIC SYMBOL	SOIL UNIT No	WEATHERING	WATER CONTENT	STRENGTH	COLOUR	FABRIC	SOIL NAME	USCS SYMBOL
	1							
	2							
	3							
	4							
	5							
	6							

	OTHER MATERIAL DESCRIPTIONS (Specify)	
7		
8		
9		

TERMINOLOGY FOR COMMON DEFECTS IN SOIL MASS

DEFECT TYPE	LAYERING (I) ✱		FRACTURES (I) ✱		CAVITIES (I) ✱	
	BEDDED/STRATIFIED(1)	LAYERED (2)(3)	JOINT/FISSURE (4)	SHEAR ZONE (5)	OPEN CAVITY (6)	INFILLED CAVITY (7)
DESCRIPTION	1) Arrangement in layers of soil particles of similar colour, size and composition 2) Arrangement of elongated or tabular particles or voids near parallel to one another or to the layers soil mass shows bedding or stratification with spacing > 100 mm	soil material is finely layered (< 25mm) or coarsely layered (25-100mm)	Single fracture across which soil has little tensile strength, planar, curved or irregular; closed open or partly infilled by soil or rock material, may be continuous over outcrop extent	Zone of multiple very closely spaced (< 25mm) fracture planes either irregular, or with roughly parallel planar boundaries that are smooth or slickensided, lens shaped blocks of soil within zone, which may be softened or wetted	Opening within soil mass, commonly tubular, may occur singly or as multiple, separate or interconnected tubes, may occur also as sheet or wall-like openings within soil mass, walls of cavity may be coated with clay/silt or organic matter	Open cavities that have been partly or completely infilled by collapsed or transported material, infilling material may be partly cemented, softened or wetted
	NOTE III Indicate bed or layer description and defect spacing as shown below				where cavities identified indicate origin (e.g. by piping)	
MAP SYMBOLS with strike and dip	081 21	32 016	'open' 060 27 'closed' 075 86	35 072 (W) W denotes presence of wetted zone or margin	SKETCH AS APPROPRIATE	

(I) OTHERS (8) Specify

DEFECT DATA SUMMARY TABLE

SOIL UNIT No	DEFECT SET No	DEFECT TYPE	DESCRIPTION (II, III): LAYERING	DESCRIPTION (II, III): FRACTURE/CAVITY	DEFECT ORIENTATION: DIP DIRECTION	DEFECT ORIENTATION: DIP VALUE	GROUND WATER	REMARKS
					±	±		
					±	±		
					±	±		
					±	±		
					±	±		
					±	±		
					±	±		
					±	±		
					±	±		

✱ NOTE I Where softened or wetted zones occur adjacent to layering, fractures or cavities, indicate on sketch and in remarks column

NOTE II Fracture/cavity description requires information on defect spacing, continuity and geometry. Sketch if appropriate

LAYERING DESCRIPTION

1	non-uniform	1: planar 2: curved 3: irregular 4: contorted
2	uniform	5: lensoidal 6: gradational 7: other (specify)

DEFECT SPACING

	TERM	SPACING (mm)
1	extremely wide	> 2000
2	very wide	500 - 2000
3	wide	200 - 500
4	moderate	100 - 200
5	close	25 - 100
6	very close	5 - 25
7	extremely close	< 5

DEFECT CONTINUITY

	TERM	LENGTH (m)
1	very high	> 10
2	high	5 - 10
3	moderate	2 - 5
4	low	0.5 - 2
5	very low	< 0.5

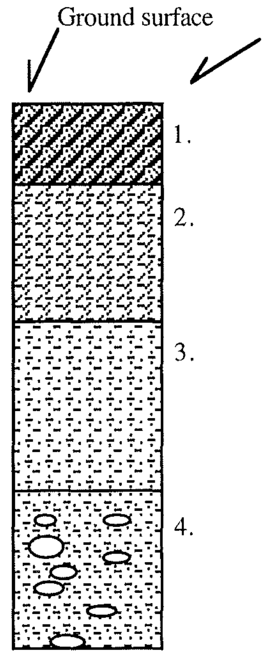
GROUNDWATER

	TERM	FLOW RATE
1	dry	
2	seepage	< 1 ml s^{-1}
3	very low flow	1 - 10 ml s^{-1}
4	low flow	10 - 100 ml s^{-1}
5	moderate flow	0.1 - 1 l s^{-1}
6	large flow	1 - 10 l s^{-1}
7	very large flow	> 10 l s^{-1}

APPENDIX 2.

A2.1 SOIL PROFILE LOGS.

Profile of Runanga Hill Soils



Soil Horizon Descriptions

Layer one. Slightly weathered, dry, firm, light greyish-brown, massive, PEATY CLAY with some silt.

Layer two. Slightly weathered, dry, firm, light greenish-brown, massive, SILTY CLAY with some peat.

Layer three. Moderately weathered, moist, soft, light greenish-yellow with brown mottles, massive SILTY CLAY.

Layer four. Highly weathered, moist, stiff, light brownish-yellow with light green-grey mottles, SILTY CLAY with some medium gravel.

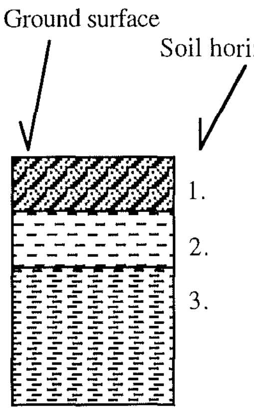
Scale 1:20

0 0.2 0.4 0.6m

(Sketch orientation East/ West)

Note:
1. The soil profile was logged from the scarp, Point Elizabeth end of North Beach. Grid reference 632655 NZMS 260 J31
2. Layer four contains highly weathered lithorelics of mudstone (parent material).
3. These soils are disturbed as a result of slope processes.
4. A distinctive contact exists between horizons 1 and 2 and gradational contacts exist between horizons 2 and 3, and between horizons 3 and 4.
5. Layers 3 and 4 were sampled for laboratory testing.

Profile of Omoto Steepland Soils

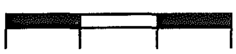


Soil Horizon Descriptions

Layer one: Unweathered, moist, spongy, black, massive, PEAT.

Layer two: Moderately weathered, wet, light yellowish-brown with light green-grey mottles, CLAY.

Layer three: Completely weathered, moist, compact, dark yellowish-brown with light greenish-grey mottles, CLAY.



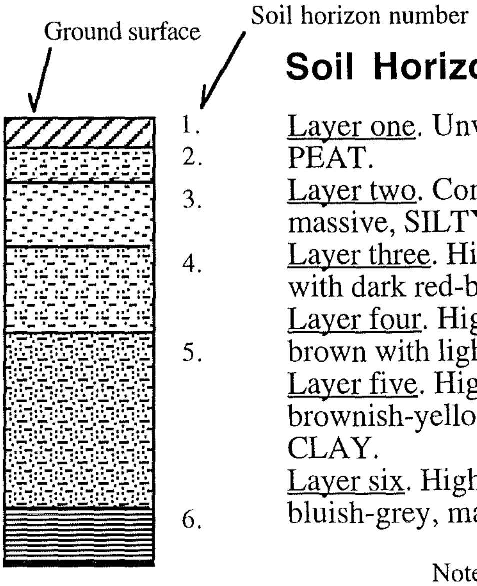
Scale 1:20

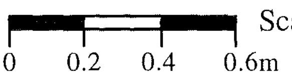
0 0.2 0.4 0.6m

(Sketch orientation North/South)

Note:
1. The soil profile was logged from Twelve Apostles Range. Grid reference 634636 NZMS 260 J31.
2. The depth of layer three is unknown.
3. Soil material from layer three becomes greasy on immersion in water.
4. All soil contacts in the profile were observed to be gradational.
5. Layers 2 and 3 were sampled for laboratory testing

Profile of Stillwater Hill Soils





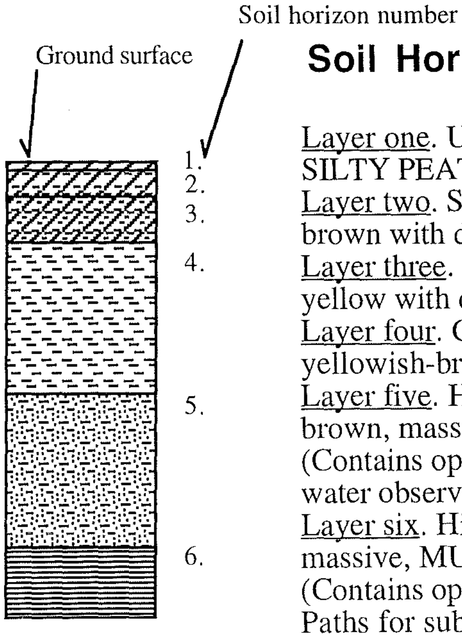
(Sketch orientation North/South)

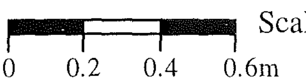
Soil Horizon Descriptions

Layer one. Unweathered, wet, spongy, black, massive, PEAT.
Layer two. Completely weathered, moist, soft, light-brown, massive, SILTY CLAY.
Layer three. Highly weathered, moist, firm, light olive-grey with dark red-brown mottles, SILTY ClAY.
Layer four. Highly weathered, moist, firm, dark yellowish-brown with light olive-grey mottles, SILTY CLAY.
Layer five. Highly weathered, moist, very soft, dark brownish-yellow with light olive-brown mottles, SILTY CLAY.
Layer six. Highly weathered, moderately weak, light bluish-grey, massive, MUDSTONE.

Note:
1. Profile location, grid reference 622556 NZMS 260 J32.
2. Small mudstone (up to 0.15m in size) lithorelics exist in layers 5, 6 and 7. The number of these increases with depth.
3. Soil horizon contacts are gradational.
4. Layer 6 was described using rock material properties.
5. Layers 3, 4 and 5 were sampled for laboratory testing.

Profile of Kaiata Hill Soils





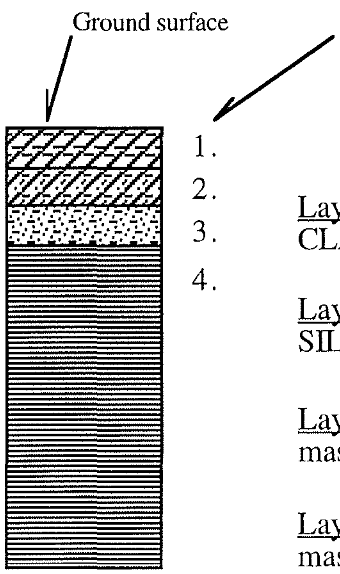
(Sketch orientation West/East)

Soil Horizon Descriptions

Layer one. Unweathered, wet, very soft, black, massive, SILTY PEAT.
Layer two. Slightly weathered, moist, firm, dark greyish-brown with dark yellow-brown mottles, CLAYEY PEAT.
Layer three. Highly weathered, moist, stiff, dark brownish-yellow with dark brownish-grey mottles, CLAYEY PEAT.
Layer four. Completely weathered, moist, hard, dark yellowish-brown, massive, CLAY.
Layer five. Highly weathered, moist, hard, light yellowish-brown, massive, SILTY CLAY.
(Contains open cavities along fractures (up to 0.1m wide), free water observed within the cavities.)
Layer six. Highly weathered, weak, light greenish-grey, massive, MUDSTONE.
(Contains open, vertical sheet cavities up to 0.15m wide. Paths for sub-surface water.)

Note:
1. Profile location Kilgour Road area. Grid reference 629593 NZMS 260 J32.
2. Layer six was classified using rock material properties.
3. Soil horizon contacts are gradational.
4. Layers 3, 4 and 5 were sampled for laboratory testing.

Profile of Kamaka Soils



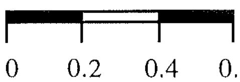
Soil Horizon Descriptions

Layer one: Unweathered, soft, light brown, massive, PEATY CLAY.

Layer two: Unweathered, moist, soft, light grey-brown, massive, SILTY PEAT with some clay.

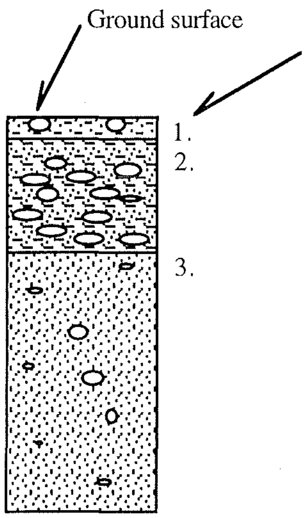
Layer three: Highly weathered, wet, firm, light blue-grey, massive, SILTY CLAY.

Layer four: Slightly weathered, dry, compact, dark blue-grey, massive, SILTY MUDSTONE.



Note:
1. Composite profile as logged from several hand auger holes.
2. Profile location Mill Creek. Grid reference 613554 NZMS 260 J32.
3. Water table was reached at depth 0.6m.
4. Hole terminated when unweathered Stillwater Mudstone reached.

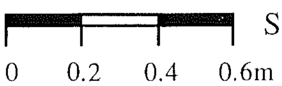
Profile of Mahinapua Soils



Soil Horizon Descriptions

Layer one. Unweathered, moist, loose, dark brownish-grey, coarsely layered, CLAYEY medium GRAVEL with some peat.
Layer two. Slightly weathered, moist, loose, dark brownish-yellow, coarsely layered, CLAYEY medium GRAVEL with some silt.

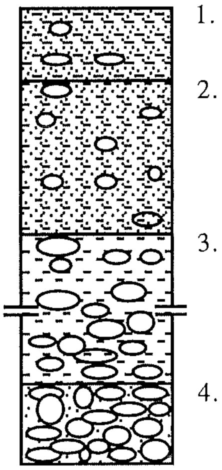
Layer three. Moderately weathered, wet, loose, dark yellowish-brown, massive, SILTY coarse SAND with some medium gravel.



(Sketch orientation East/West)

Note:
1. Profile location Point Elizabeth end of North Beach. Grid reference 631658 NZMS 260 J31.
2. Lower horizons are heavily iron stained.
4. All contacts between soil horizons are gradational.
5. No water table was observed.

Profile of Rutherglen Soils



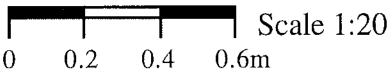
Soil Horizon Descriptions

Layer one. Unweathered, moist, loose, dark brownish-yellow, massive, CLAYEY medium SAND with some medium gravel.

Layer two. Moderately weathered, wet, loose, light brownish-yellow, massive, medium SANDY CLAY with some medium gravel.

Layer three. Moderately weathered, wet, loose, dark yellowish-brown, massive, medium-coarse Gravel with some clay.

Layer four. Moderately weathered, moist, cemented, dark yellowish-red, massive, medium- coarse Gravel with some silt.



(Sketch orientation North/South)

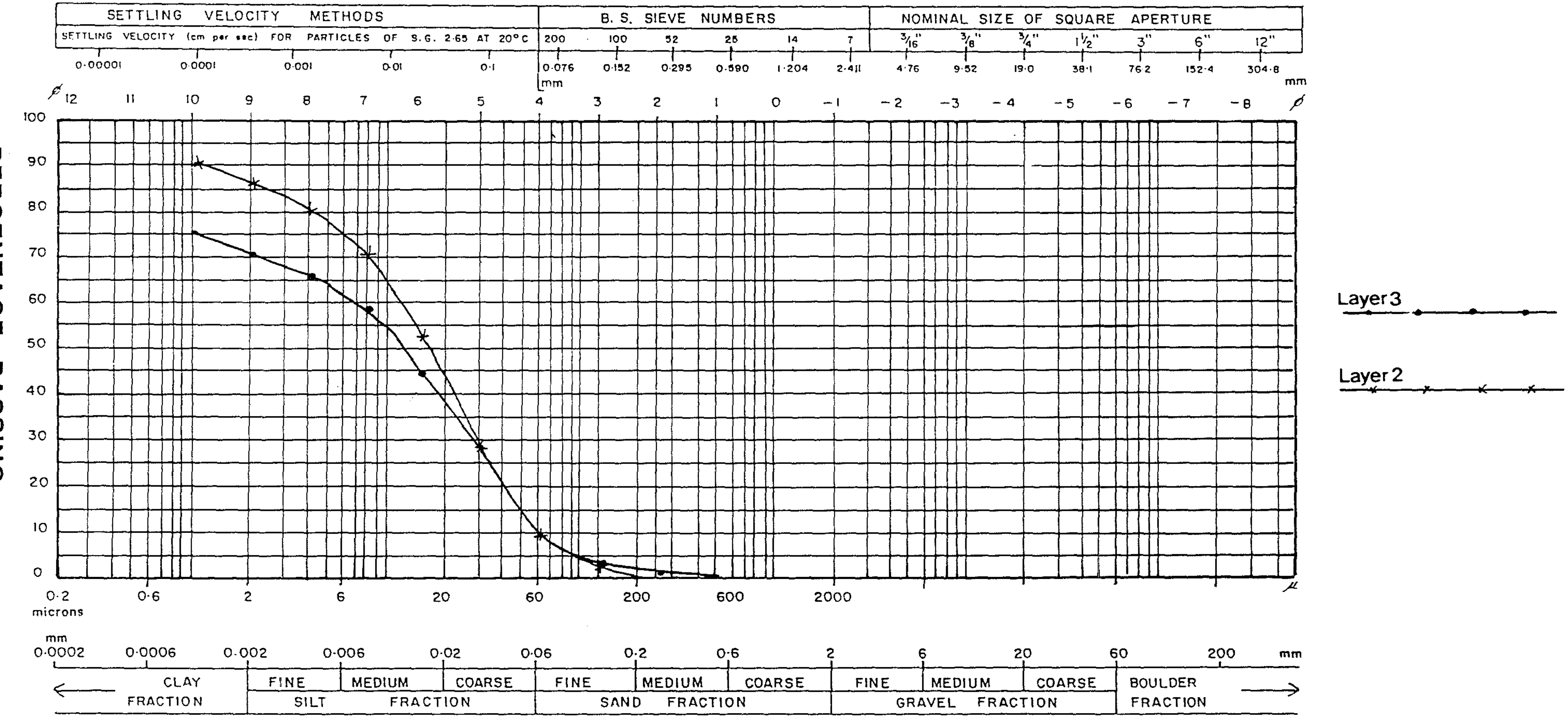
Note: 1. Total depth soil horizon 3 is 2m.
2. Soil horizon 4 is presumed to be the parent material of the Mahinapua Soils.

A2.2 PARTICLE SIZE TEST RESULTS.

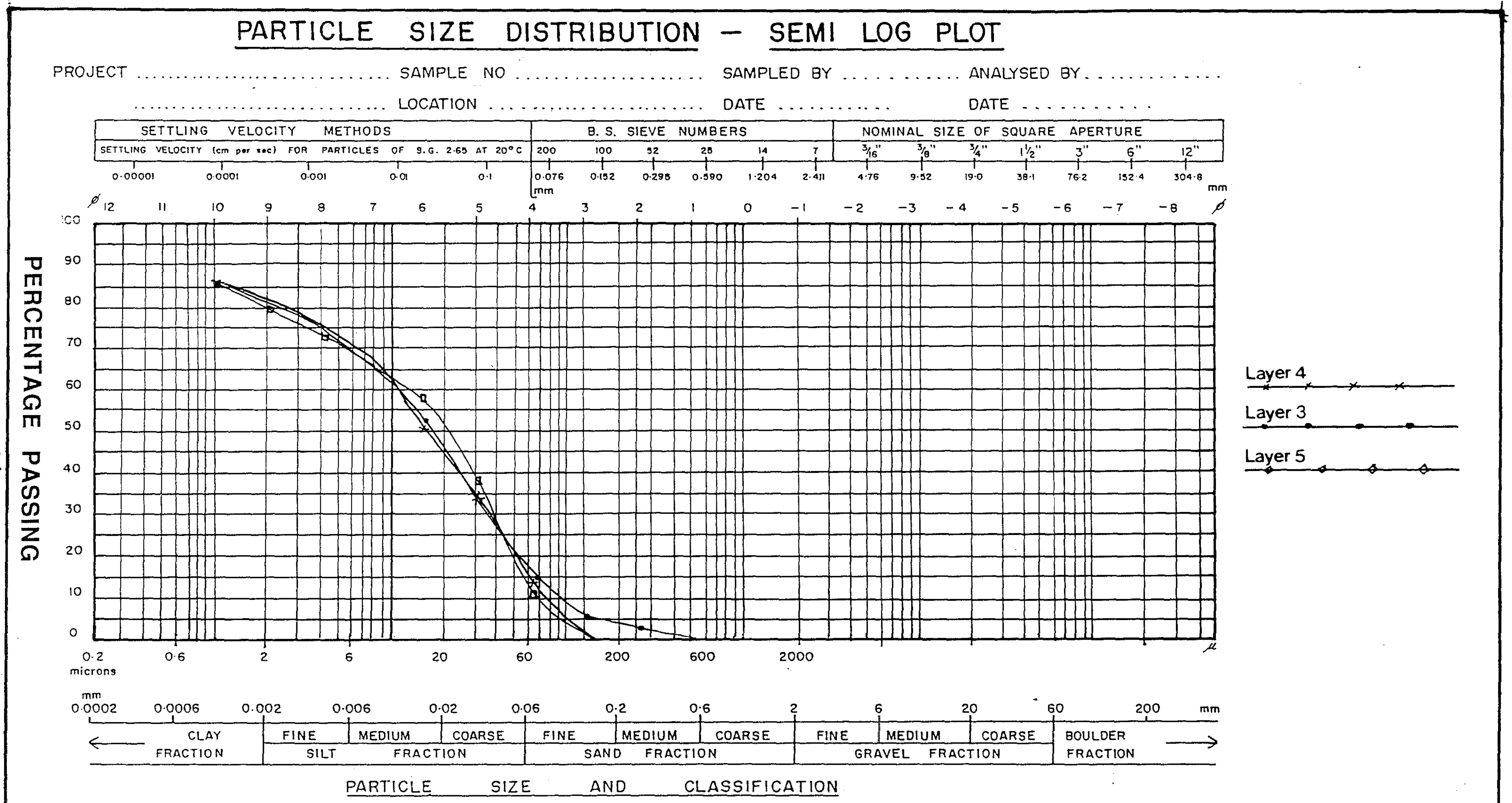
PARTICLE SIZE DISTRIBUTION — SEMI LOG PLOT

PROJECT .... SAMPLE NO .... SAMPLED BY .... ANALYSED BY ....

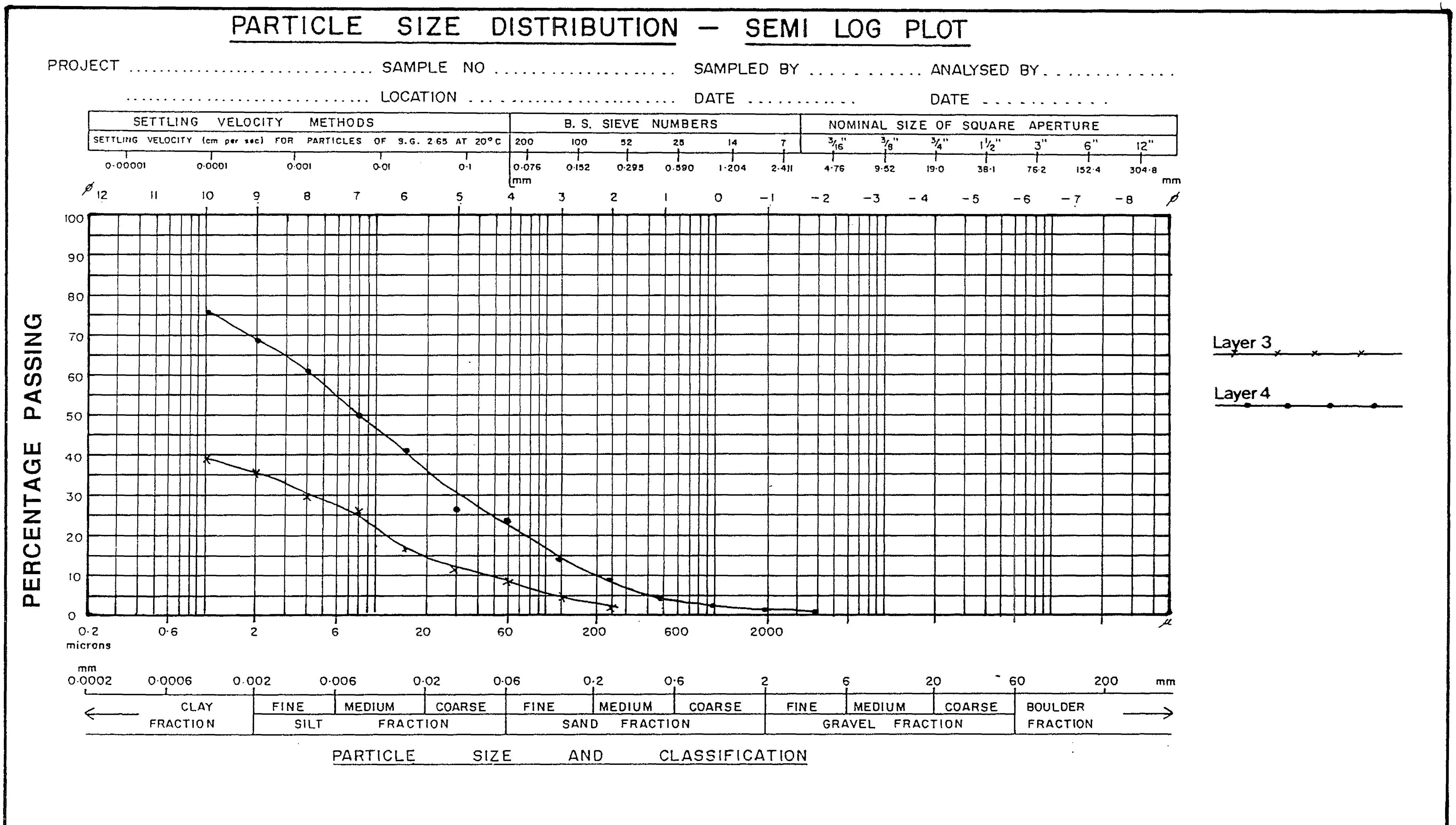
.... LOCATION .... DATE .... DATE ....



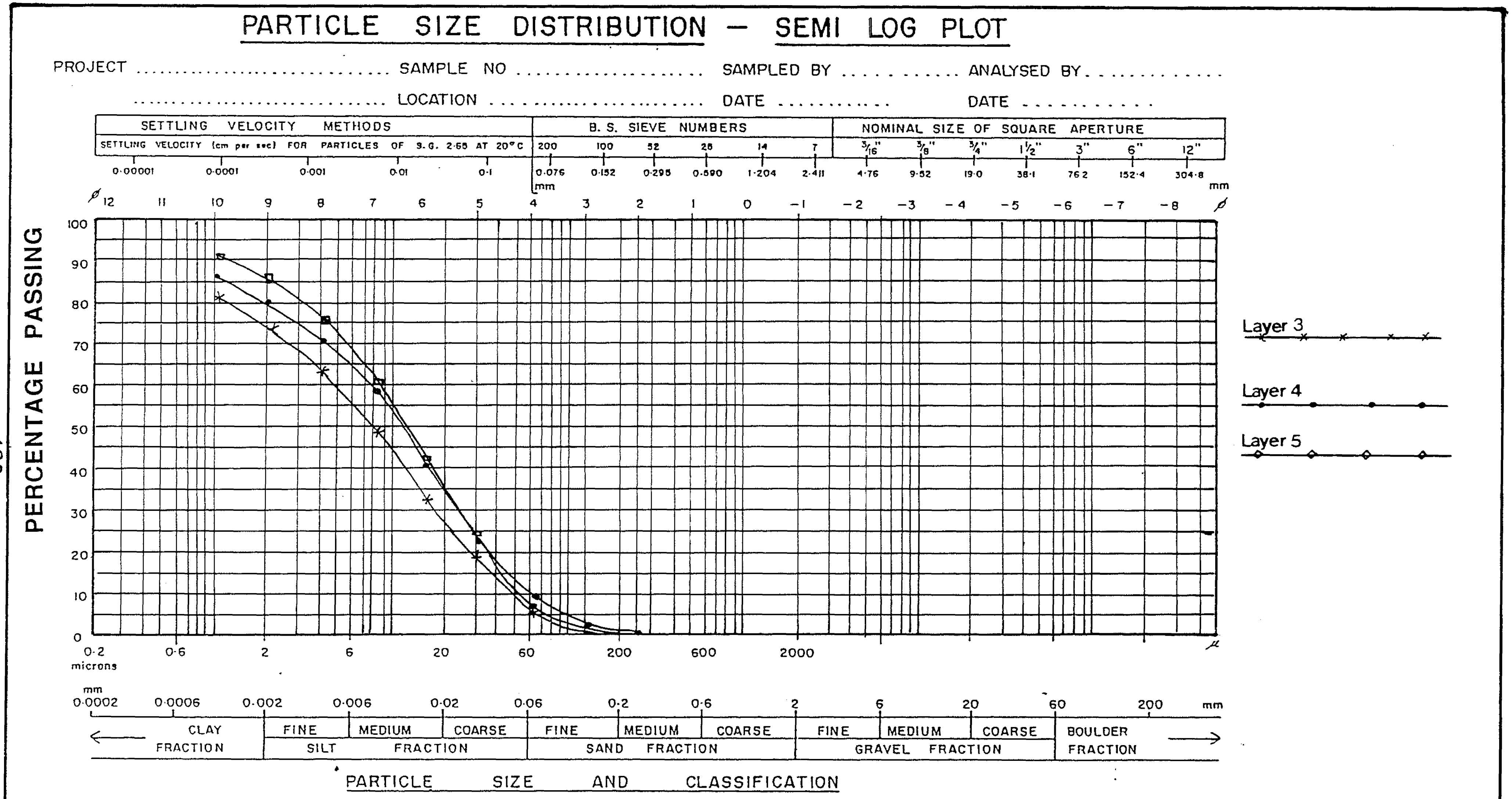
OMOTO STEEPLAND SOILS



KAIATA HILL SOILS

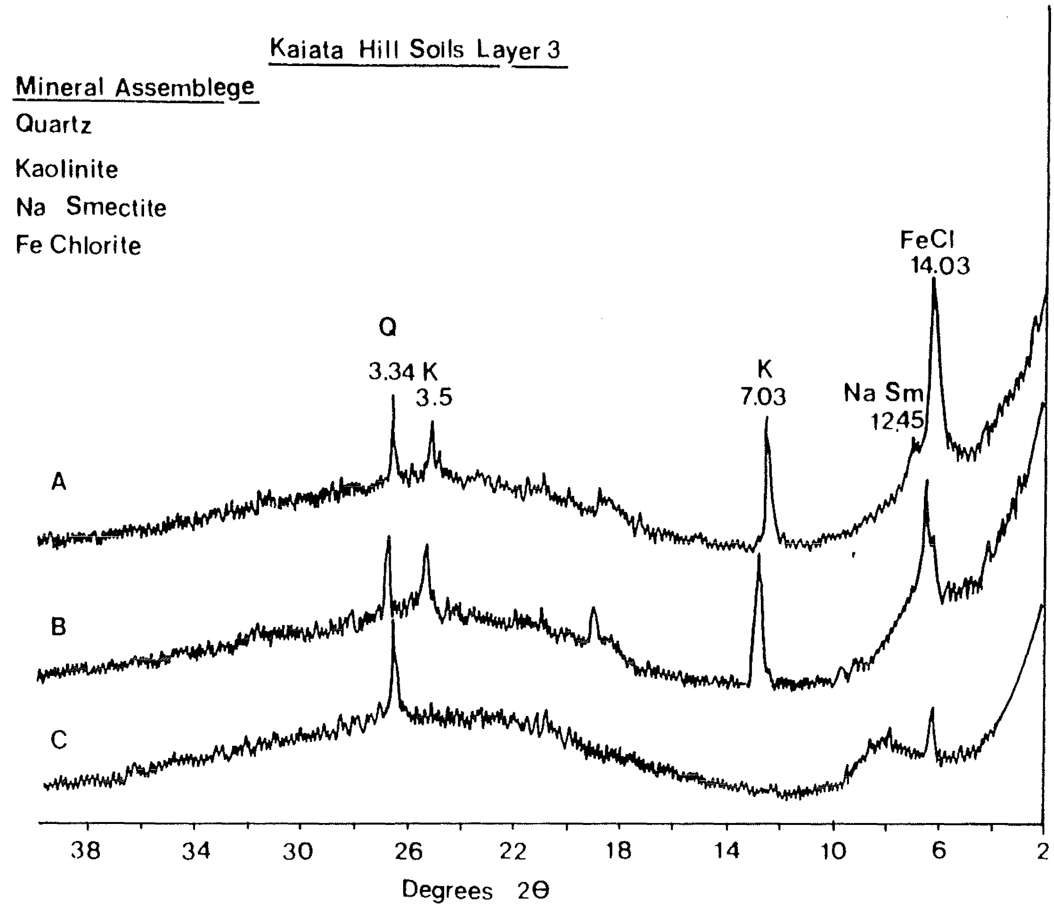


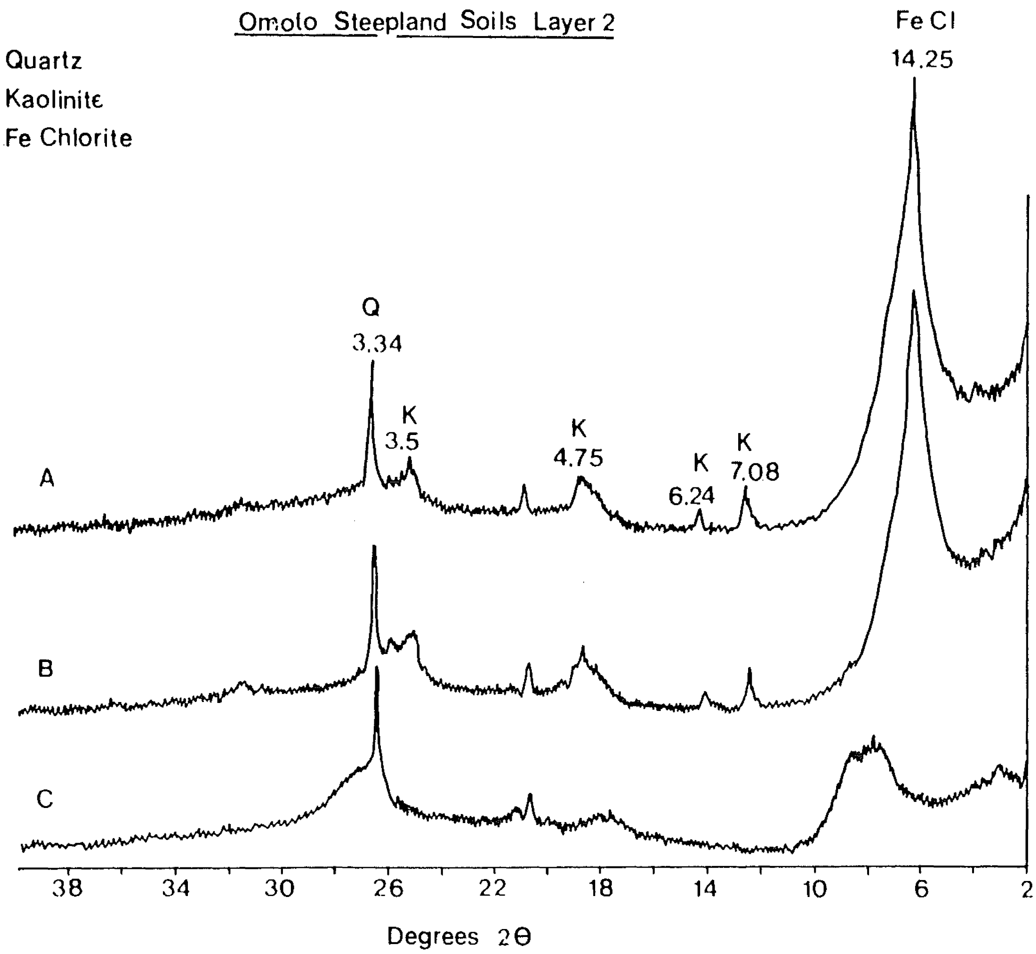
RUNANGA MATRIX SAMPLES

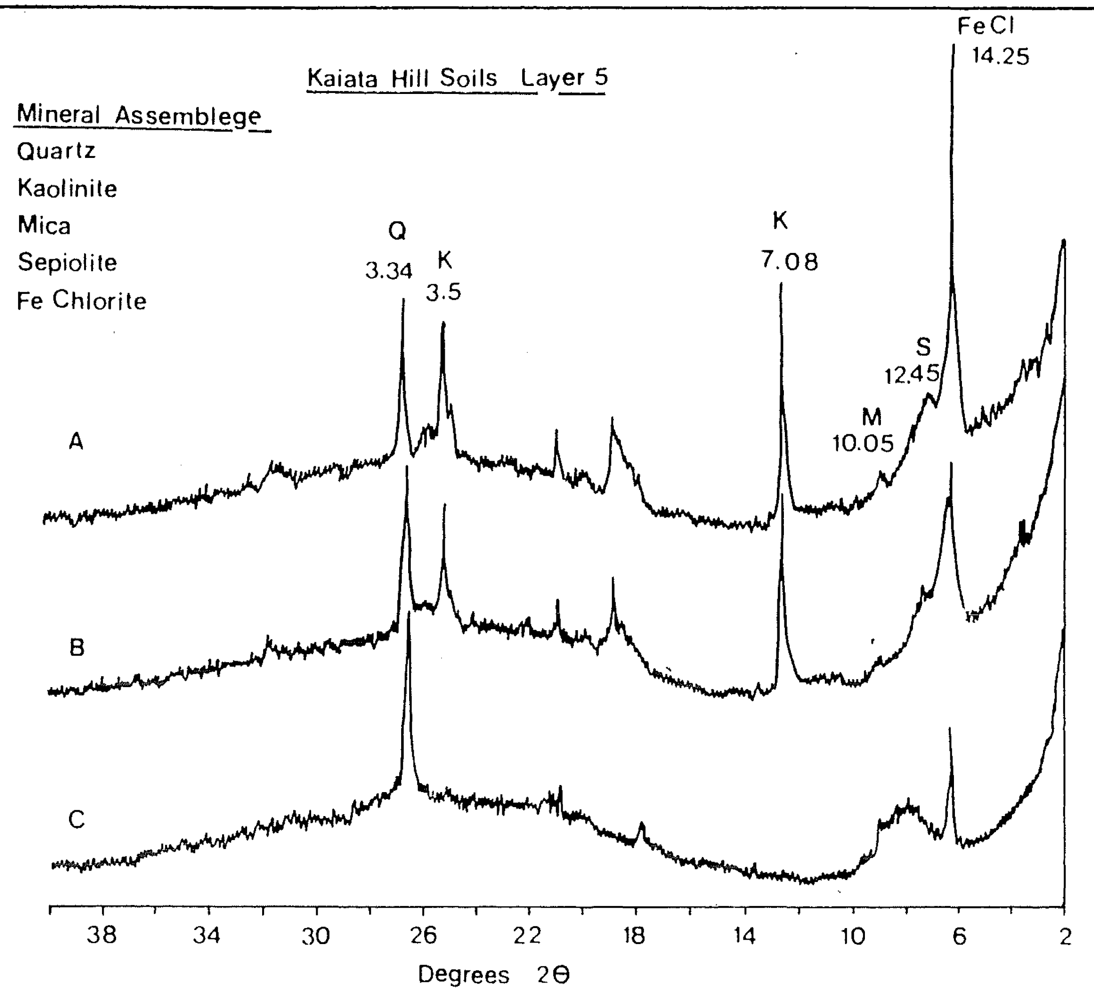


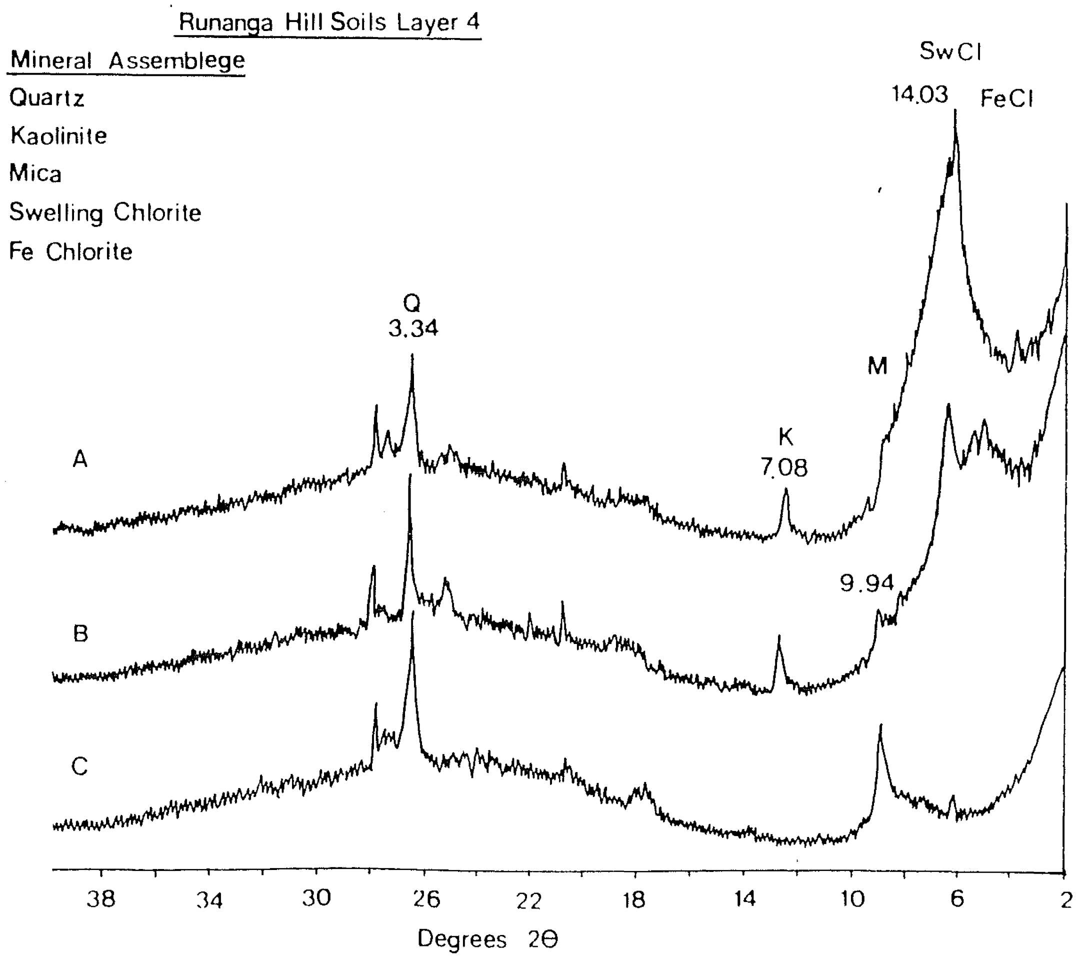
STILLWATER MATRIX SAMPLES

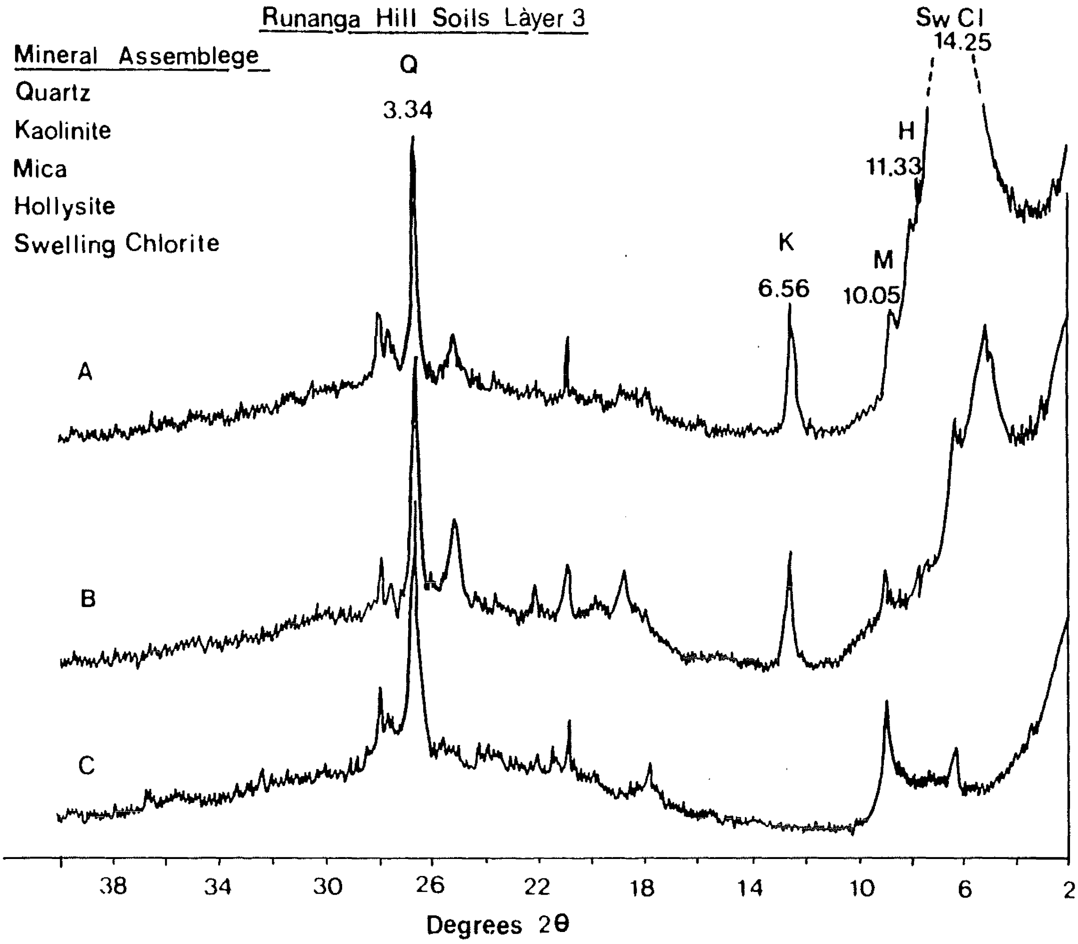
A2.3 CLAY MINERALOGY GRAPHS.

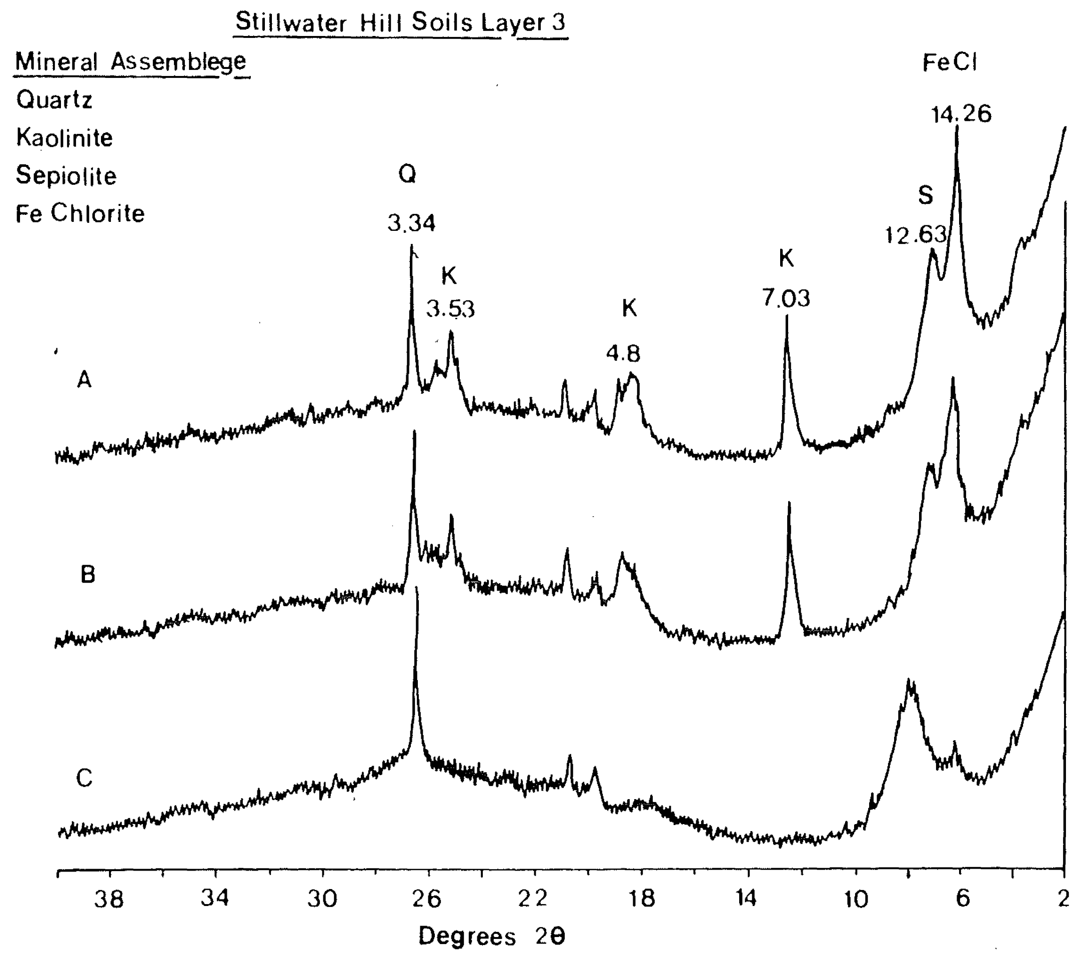





Kaiata Hill Soils Layer 5
Mineral Assemblege
Quartz
Kaolinite
Mica
Sepiolite
Fe Chlorite
Q
3.34
K
3.5
K
7.08
S
12.45
M
10.05
FeCl
14.25
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2Θ


Runanga Hill Soils Layer 4
Mineral Assemblege
Quartz
Kaolinite
Mica
Swelling Chlorite
Fe Chlorite
SwCl
14.03
FeCl
Q
3.34
M
K
7.08
9.94
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2Θ


Runanga Hill Soils Layer 3
Mineral Assemblege
Quartz
Kaolinite
Mica
Hollysite
Swelling Chlorite
Q
3.34
K
6.56
M
10.05
H
11.33
Sw Cl
14.25
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2θ


Stillwater Hill Soils Layer 3
Mineral Assemblege
Quartz
Kaolinite
Sepiolite
Fe Chlorite
Q
3.34
K
3.53
K
4.8
K
7.03
S
12.63
FeCl
14.26
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2θ

Stillwater Hill Soils Layer 4

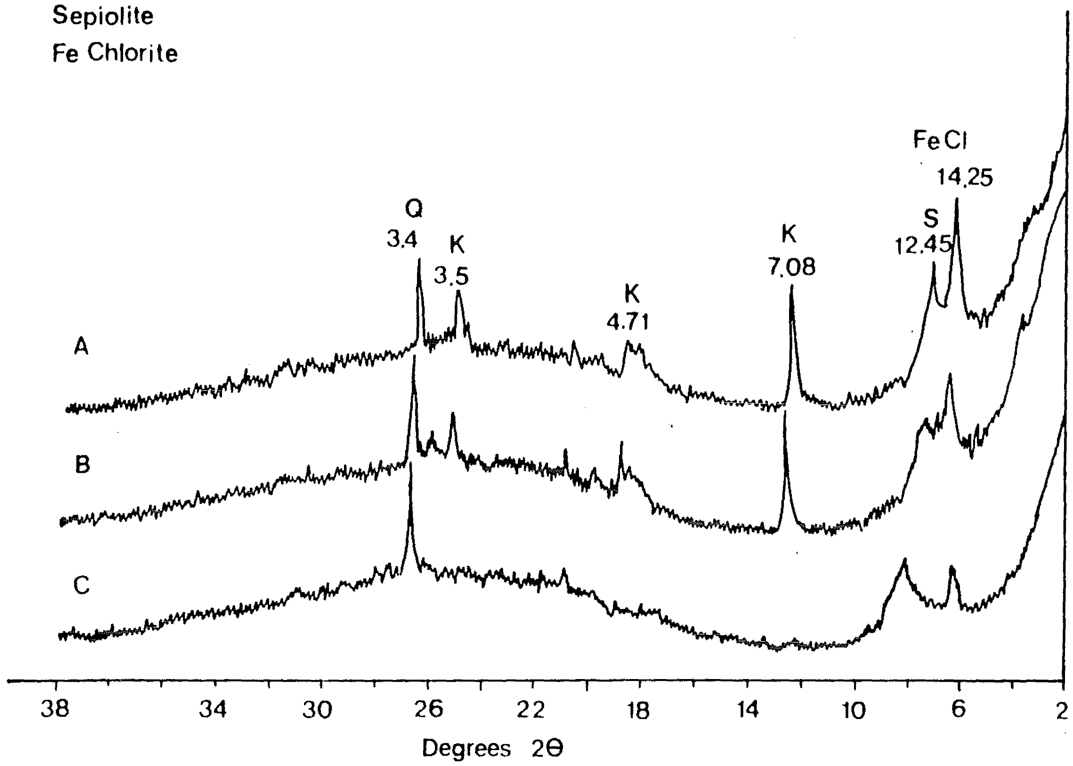
Mineral Assemblege

Quartz

Kaolinite

Sepiolite

Fe Chlorite



Stillwater Hill Soils Layer 5

Mineral Assemblege

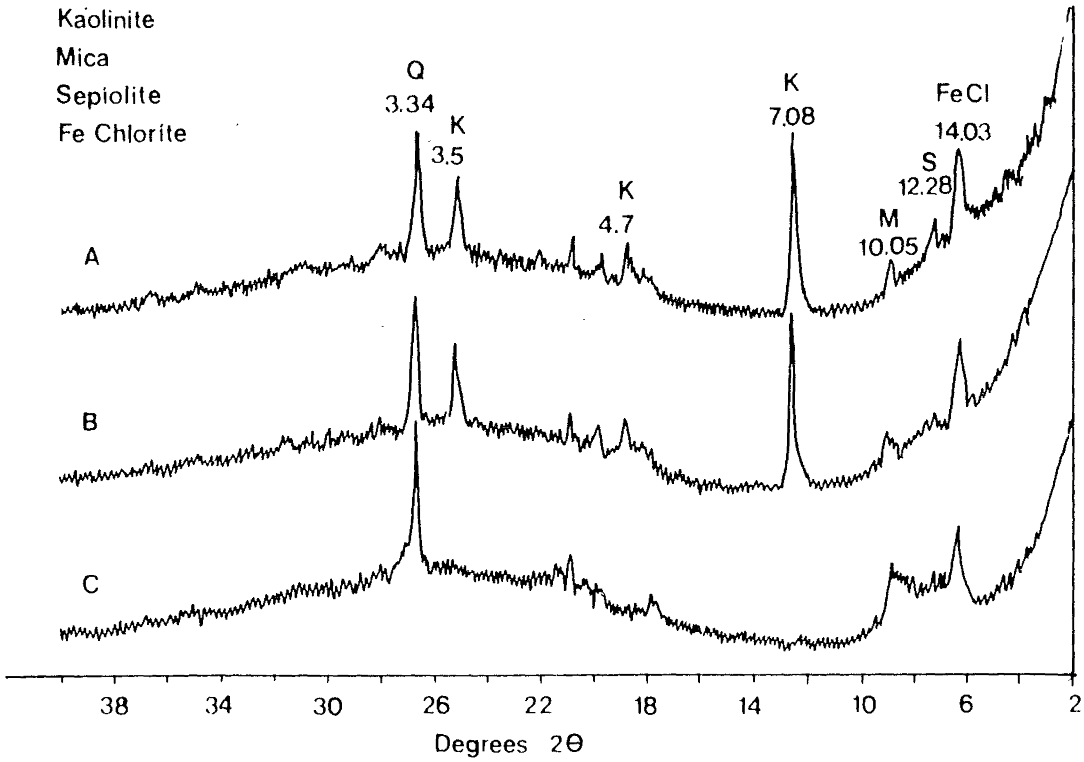
Quartz

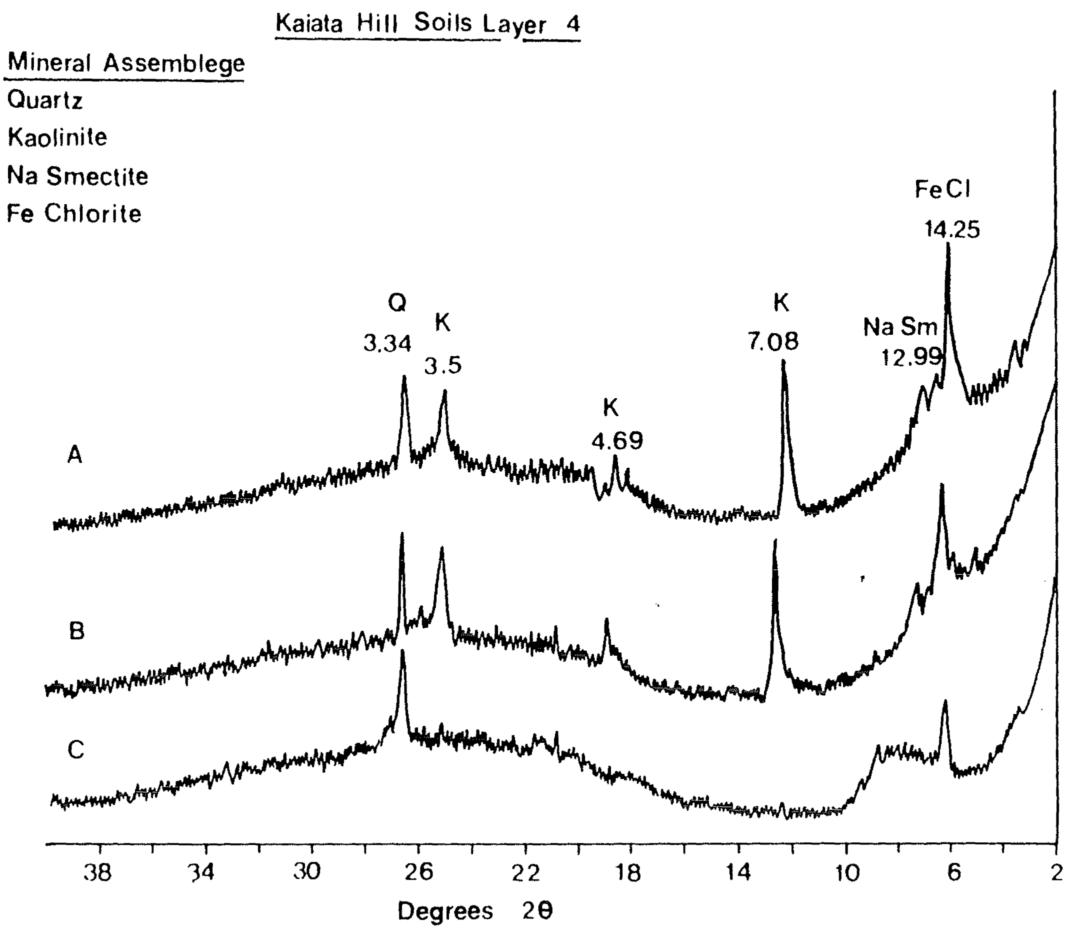
Kaolinite

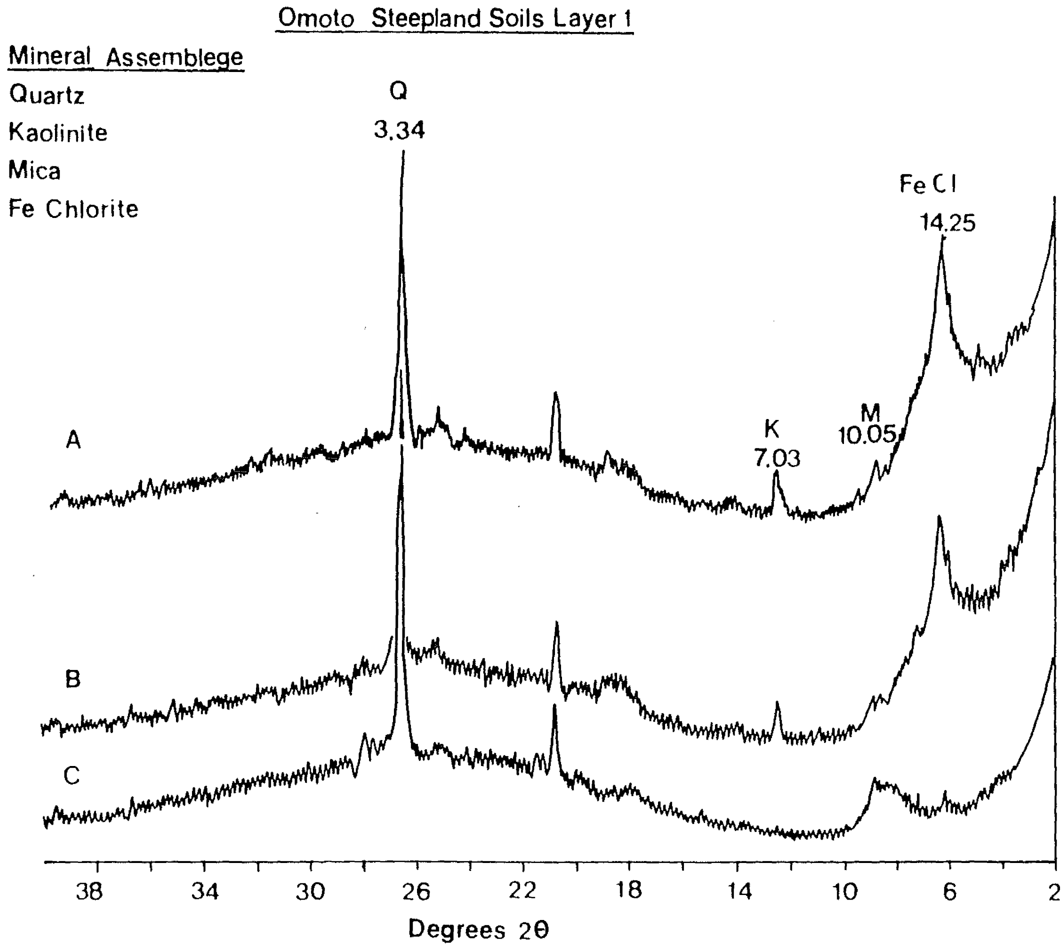
Mica

Sepiolite

Fe Chlorite




Kaiata Hill Soils Layer 4
Mineral Assemblege
Quartz
Kaolinite
Na Smectite
Fe Chlorite
Q
3.34
K
3.5
K
4.69
K
7.08
Na Sm
12.99
FeCl
14.25
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2θ

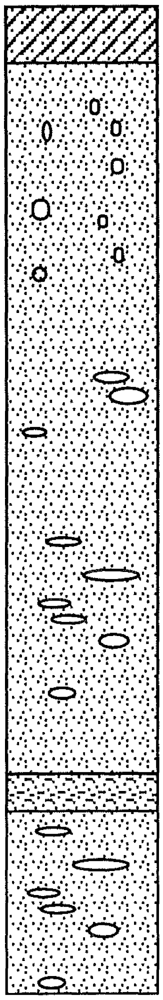

Omoto Steepland Soils Layer 1
Mineral Assemblege
Quartz
Kaolinite
Mica
Fe Chlorite
Q
3.34
K
7.03
M
10.05
Fe Cl
14.25
A
B
C
38
34
30
26
22
18
14
10
6
2
Degrees 2θ

APPENDIX 3.

A3.1 STANTON CRESCENT AUGER HOLE LOGS.

Auger Hole 1

Profile descriptions



1. Layer one: Unweathered, moist ,soft, light brown, massive, medium SANDY PEAT with some fine gravel.

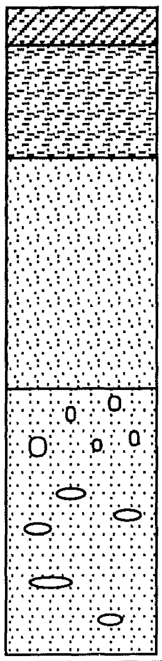
2. Layer two: Slightle weathered, moist, loose, light yellowish grey, massive, fine GRAVELLY medium SAND.

3. Layer three: Highly weathered, dry, crumbly, massive, dry yellowish brown CLAY/SILT.

4. Layer four: Slightle weathered, moist, loose, light yellowish grey, massive, fine GRAVELLY medium SAND.

Scale 1:20

Auger Hole 3



1. Layer one: Moderatelt weathered, moist, spongy, light greyish brown, massive, CLAYEY PEAT.

2. Layer two: Moderately weathered, moist, soft, light yellowish green, massive, medium SANDY CLAY.

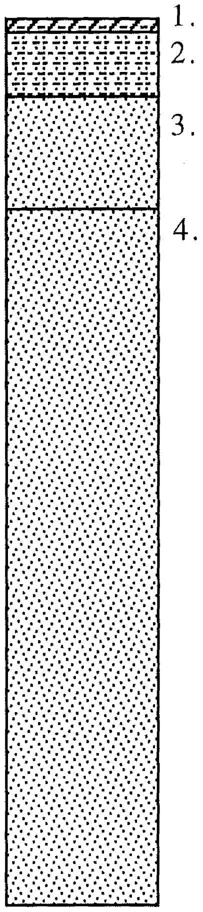
3. Layer three: Moderately weathered, moist, soft, light yellowish green, massive, medium SAND.

4. Layer four: Moderately weathered, moist, loose, light greenish yellow, massive, medium GRAVELLY medium SAND.

Note: Profile locations are shown on Figure 4.6. See text for further explanation.

Profile description.

Auger Hole 2

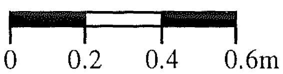


Layer One: Unweathered, moist, spongy, light greyish brown, massive, CLAYEY PEAT.

Layer Two: Moderately weathered, moist, firm, light greenish grey, iron mottled, SILTY CLAY.

Layer Three: Moderately weathered, moist, loose, yellowish brown, iron mottled, medium SAND.

Layer Four: Moderately weathered, moist, loose, light greenish grey, medium SAND.

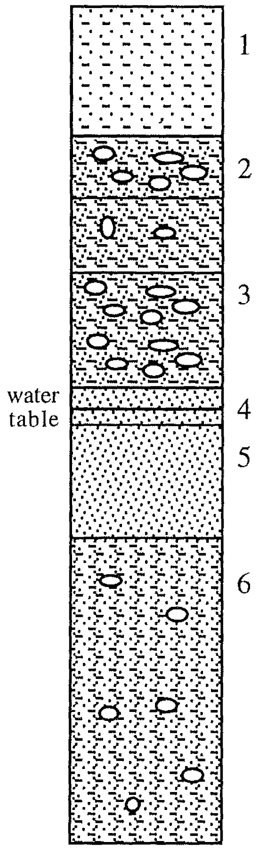


Scale 1:20

A3.2 AUSTRALASIAN HOTEL SLIDE AUGER HOLE LOGS.

@ 20m from fence

Profile descriptions.



Layer one: Unweathered, Wet, very soft, dark brownish grey, massive, CLAYEY SILT.

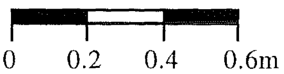
Layer two: moderately weathered, wet, soft, light brownish grey, massive, CLAYEY medium GRAVEL.

Layer three: moderately weathered, wet, soft, light blueish grey, massive, fine GRAVELLY medium SAND with some clay.

Layer four: slightly weathered, wet, very soft, brownish grey, massive, medium SANDY SILT.

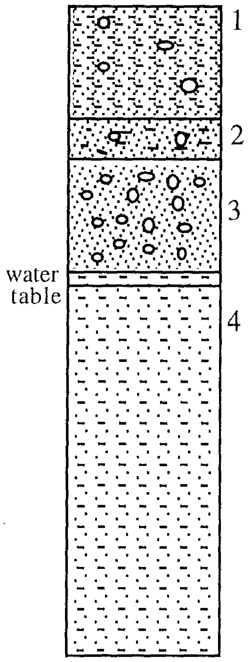
Layer five: moderately weathered, saturated, very soft, light blueish grey, massive, SANDY SILT.

Layer six: moderately weathered, wet, very soft, light greyish blue, massive, medium SANDY CLAY with some fine gravel.



Scale 1:20

@ 40m from fence

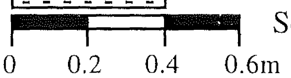


Layer one: moderately weathered, moist, firm, dark brownish grey, iron mottled, medium SANDY CLAY with some fine gravel.

Layer two: moderately weathered, wet, very soft, light brownish grey, massive, medium GRAVELLY medium SAND with some clay.

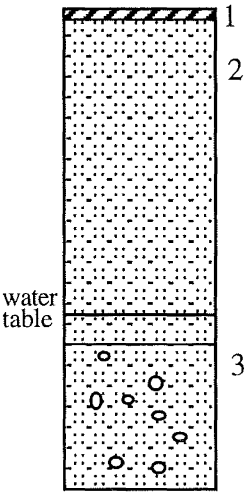
Layer three: moderately weathered, wet, loose, dark brown, massive, medium SANDY fine GRAVEL.

Layer four: highly weathered, wet, firm, light blueish grey, massive, CLAYEY SILT.



Scale 1:20

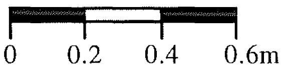
@ 60m from fence



Layer one: Peat

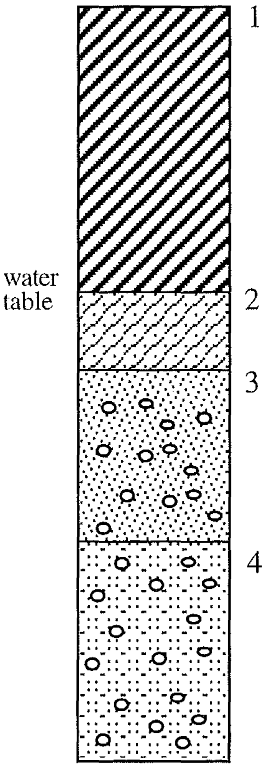
Layer two: Moderately weathered, wet, soft, light brownish grey, iron mottled, medium SANDY CLAY.

Layer three: Moderately weathered, wet, soft, light brownish grey, iron mottled, medium SANDY CLAY with some fine gravel.



Scale 1:20

@ 80m from fence

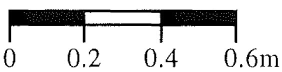


Layer one: Moderately weathered, moist, firm, dark brown, massive, medium SANDY PEAT.

Layer two: Slightly weathered, wet, very soft, dark brown, mottled, PEATY SAND.

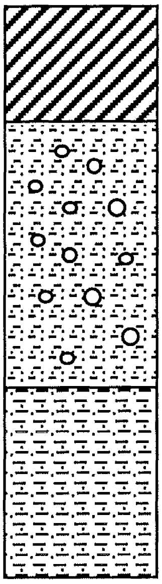
Layer three: Moderately weathered, saturated, very soft, light yellowish grey, iron mottled, fine GRAVELLY medium SAND.

Layer four: Highly weathered, moist, firm, light blue, massive, medium SANDY CLAY with some fine gravel.



Scale 1:20

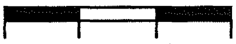
@ 100m from fence



1 Layer one: Slightly weathered, moist, soft, dark brownish grey, massive, medium SANDY PEAT.

2 Layer two: Moderately weathered, wet, soft, dark brownish grey, massive, CLAYEY medium SAND with some fine gravel.

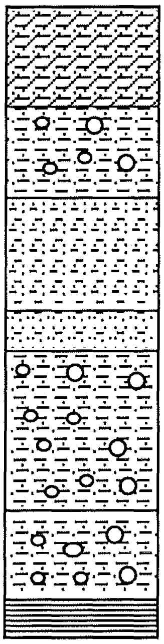
3 Layer three: Highly weathered, moist, firm, light blueish grey, massive, SILTY CLAY.



0 0.2 0.4 0.6m

Scale 1:20

@ 120m from fence



water table

1 Layer one: Slightly weathered, moist, very soft, light brown, massive, medium SANDY CLAY with some peat. Grades into Layer two.

2 Layer two: moderately weathered, moist, firm, light brownish blue, iron mottled, CLAYEY medium SAND with some fine gravel.

3 Layer three: Moderately weathered, wet, very soft, light brownish grey, massive, CLAYEY medium SAND with some silt.

4 Layer four: Slightly weathered, wet, very soft, light brownish yellow, iron mottled, SILTY medium SAND with some clay.

5 Layer five: Moderately weathered, wet, firm, light yellowish grey, iron mottled, medium SANDY CLAY with some fine gravel.

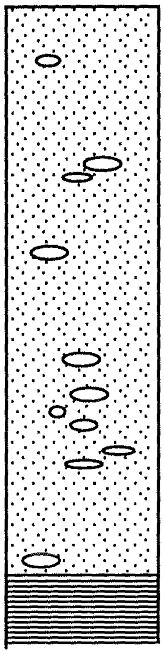
6 Layer six: Slightly weathered, moist, stiff, dark greenish blue, iron mottled, medium SANDY CLAY with some fine gravel.

7 Layer seven: Moderately weathered, moderately strong, dark bluish grey, massive, Calcareous MUDSTONE.

0 0.2 0.4 0.6m

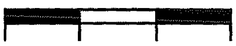
Scale 1:20

@ 140m from fence



1 Layer one: Slightly weathered, moist, loose, dark brownish yellow, iron mottled, medium SAND with some gravel.

2 Layer two: Moderately weathered, moderately strong, dark bluish grey, massive, Calcareous MUDSTONE.



0 0.2 0.4 0.6m

Scale 1:20

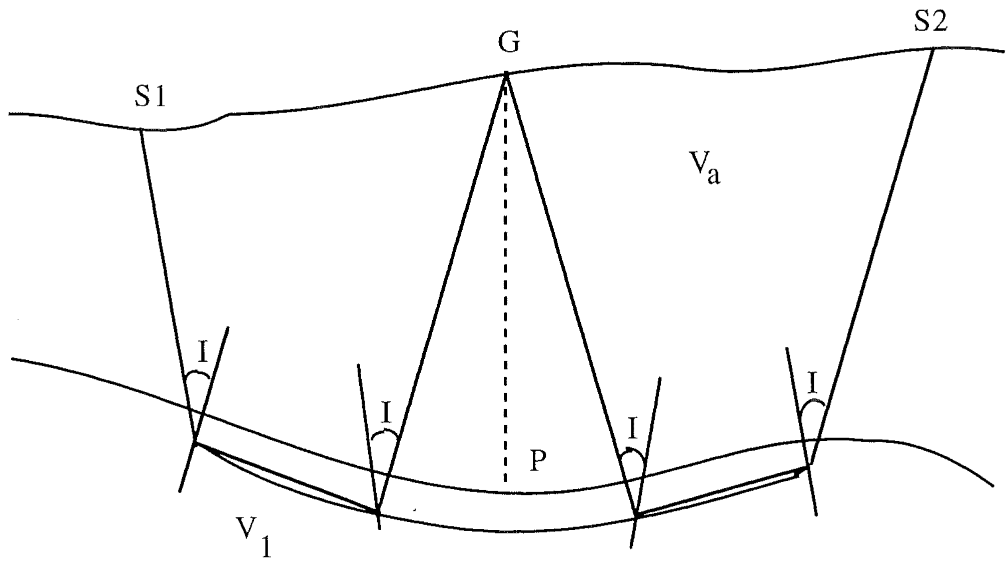
A3.3 SEISMIC REFRACTION.

A3.3.1 INTRODUCTION.

The seismic refraction exercise was undertaken in an effort to determine the depth to bedrock on the landslide surface.

A3.3.2 SEISMIC REFRACTION THEORY.

The principle of seismic refraction relies on an extension of Snell's law in optics. A seismic signal is induced through the ground by striking a metal plate with a hammer which also initiates timing circuitry illustrated in Figure A3.1 below. A single geophone at some distance from the source detects the first arrival of the seismic signal and the time between the generation and the arrival of the wave is then recorded. This process is repeated for various hammer point-geophone distances. Once the maximum distance has been reached the process is reversed and the survey repeated in the opposite direction. This ensures that in all areas where overburden thicknesses are changing the true velocity of bedrock may be calculated (Bullock 1978).



S2 G = reverse critical ray path to geophone (G)
S1 G = forward critical ray path to geophone (G)
S1 S2 = total critical ray path for survey

Figure A3.1 Time-depth definition diagram redrawn from Hawkins (1961).

The seismic velocity of the overburden is obtained from a graph of seismic wave velocity. The "time-depth" to the refractor allows calculation of the actual seismic velocity of the bedrock. The time depth to a refractor is defined as by Hawkins (1961) as "the time delay associated with the critical ray in travelling between the refractor and the surface" illustrated in Figure A3.2.

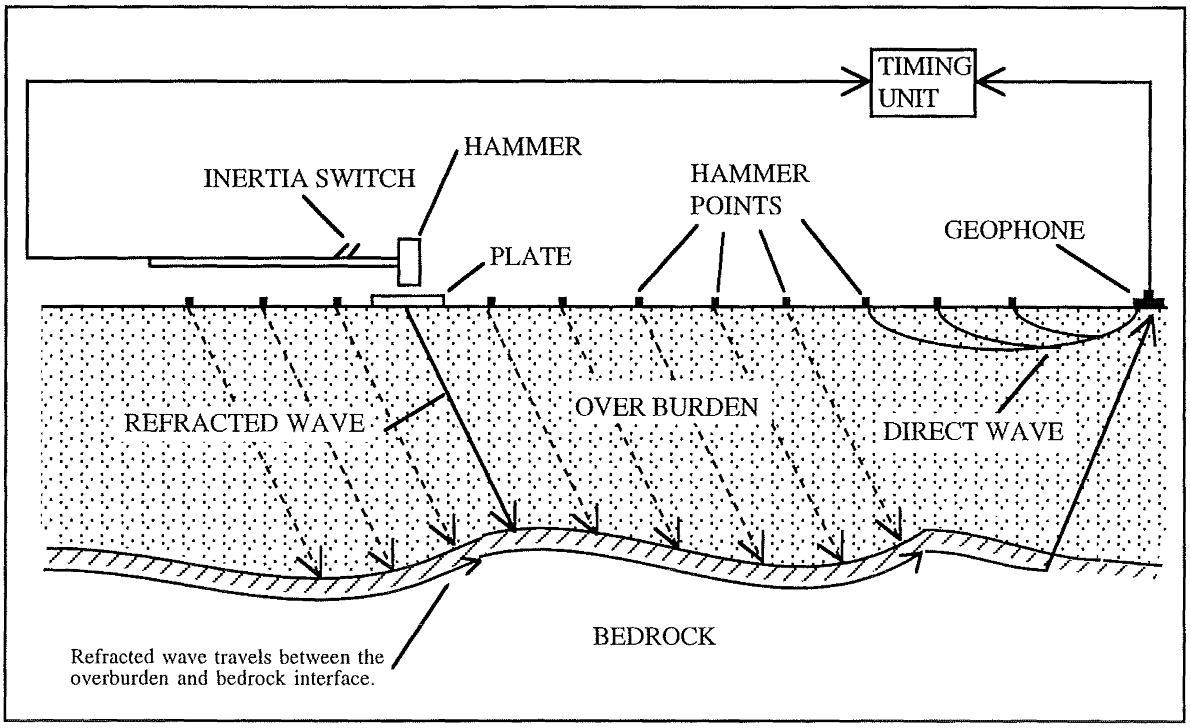


Figure A3.2 The principle of the two layer case hammer seismic method. Redrawn from Bullock (1978).

The true seismic velocity of both the bedrock and surficial material is obtained from a corrected Time Distance graph. This then allows the depth conversion factor and the radial depth to the bedrock to be determined. The bedrock profile may then be plotted.

A3.3.3 METHODOLOGY.

A shallow seismic refraction profile using the Soil Test Incorporated (model MD-9A) single channel seismic refraction unit was used to obtain continuous subsurface information on the depth to bedrock on the Australasian Hotel Slide. A Wild Distomat DI1000 EDM was used to survey the ground surface. The energy source for the survey was a steel plate and a 30 kg hammer. Travel times for forward and reverse traverses were recorded and then analysed using the reciprocal method of Hawkins (1961).

Dense vegetation on the surface of the slide made it necessary to clear a track with a slasher. This was kept to a minimum to avoid destabilising the slope.

Four separate traverses were necessary to cover the slope, three of 50m in length and one of 30m. The first traverse (0-50m) commenced at the toe of the slope. Each subsequent traverse overlapped the previous by 10m to ensure complete coverage of the slide in all areas where the seismic velocity may be changing. Initial geophone spacings at the start of each profile were close so as to ensure that V_0 intersected the origin. Correlation of the seismic profile was attained through the auger holes put down along the length of the slide.

A3.3.4 RESULTS.

The results and full calculations, time distance graphs and plotted bedrock profiles are illustrated in the following pages. Minor variations in the velocity sections of the corrected Time Distance graphs can be noticed. It was therefore necessary to draw best fit lines for individual velocity sections. The results are discussed in Section 4.5.4.

Results and calculations of seismic refraction traverse 0 - 50m.

Station	R.L.	T_f	T_r	T_d	$T_f - T_d$	$T_r - T_d$	F	d
0	14.0	57.5	0.0	-0.05	57.55	0.05		
1	14.2	53.5	3.5	-0.3	53.8	3.8		
2	14.6	51.5	8.6	1.25	50.25	7.35		
4	15.6	50.6	16.1	4.55	46.05	11.55	277.3	1.261
6	16.5	48.6	25.1	8.05	40.55	17.05	277.3	2.23
10	18.3	46.2	29.8	9.2	37.0	20.6	277.3	2.55
14	19.8	43.6	32.6	9.3	34.3	23.3	277.3	2.58
18	20.9	41.6	33.0	8.5	33.1	24.5	277.3	2.4
22	22.2	14.2	17.2	-13.1	27.3	30.3	277.3	3.6
26	23.1	32.4	30.5	2.65	29.75	27.85	277.3	0.7
30	24.2	29.2	33.7	2.65	26.55	33.1	277.3	0.859
38	26.3	23.8	41.0	3.6	20.2	37.4	307	1.1
42	27.5	21.4	54.9	9.35	12.05	45.55	307	2.87
46	28.8	13.5	55.0	5.45	8.05	49.55	307	1.67
48	29.4	6.8	55.8	2.5	4.3	53.3		
50	30.4	0.0	57.6	0.0	0.0	57.6		

Results and calculations of seismic refraction traverse 40 - 90m.

Station	R.L.	T_f	T_r	T_d	$T_f - T_d$	$T_r - T_d$	F	d
40	26.9	40.35	0.0	0.05	40.3	-0.05		
41	27.2	39.8	4.5	2.025	37.775	2.475		
42	27.5	38.5	8.0	3.125	35.375	4.875		
44	28.2	37.5	16.6	6.925	30.575	9.675	252	1.74
46	28.8	38.0	18.4	8.075	29.925	10.325	252	2.04
50	30.4	36.7	19.6	8.025	28.675	11.575	252	2.04
54	31.1	33.8	20.4	6.975	26.825	13.425	252	1.76
58	31.8	32.6	23.5	7.925	24.675	15.575	252	2.0
62	32.9	30.1	24.4	7.125	22.975	17.275	252	1.8
66	33.9	28.9	27.6	8.125	20.775	19.475	252	2.1
70	35.0	26.2	28.8	7.375	18.825	21.425	252	1.9
74	36.3	21.6	31.7	6.525	15.075	25.175	254.4	1.7
78	37.4	19.4	32.9	6.025	13.375	26.875	254.4	1.5
82	38.7	15.4	36.9	6.025	9.375	30.875	254.4	1.5
86	39.9	14.6	39.1	6.725	7.875	32.375	254.4	1.7
88	40.5	7.1	39.7	3.275	3.825	36.425		
90	41.1	0.0	40.15	-0.05	0.05	40.2		

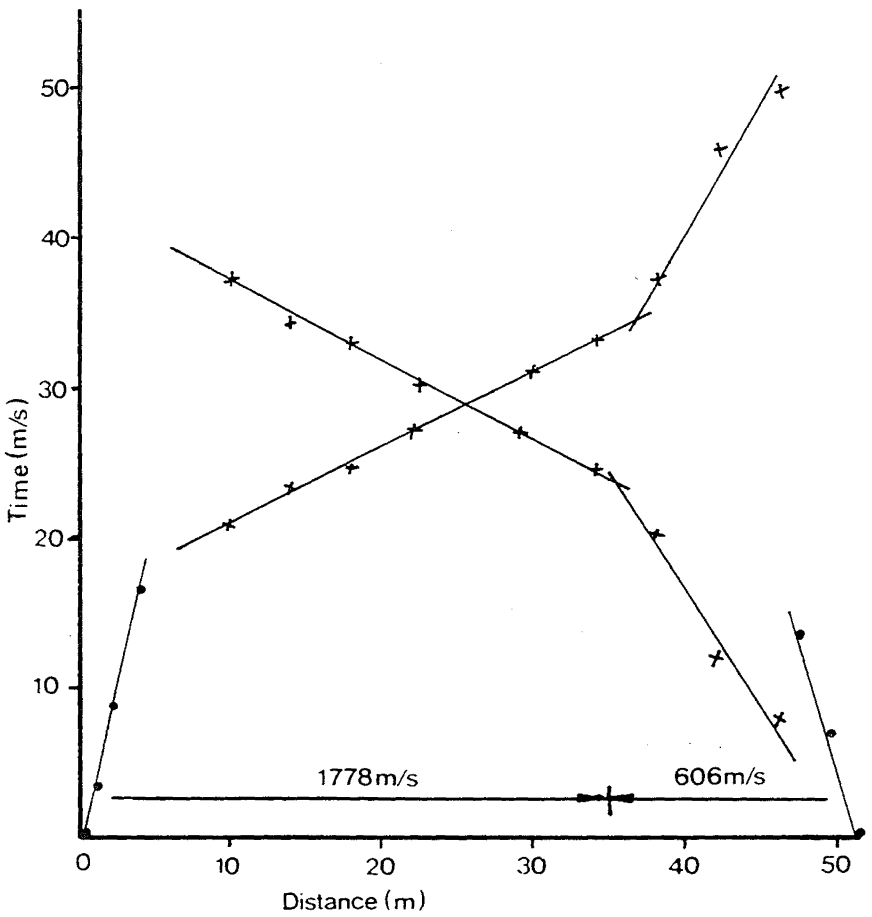
Results and calculations of seismic refraction traverse 80 - 130m.

Station	R.L.	T_f	T_r	T_d	$T_f - T_d$	$T_r - T_d$	F	d
80	38.1	35.2	0.0	-0.225	35.425	0.225		
81	38.3	34.4	3.9	1.325	33.075	2.575		
82	38.7	33.4	7.4	2.575	30.825	4.825		
84	39.3	32.0	13.1	4.725	27.275	8.375	279	1.32
86	39.9	30.2	14.0	4.275	25.925	9.725	279	1.2
90	41.1	25.6	18.2	4.075	21.525	14.125	279	1.1
94	42.5	22.6	24.0	5.475	17.125	18.525	279	1.5
98	43.7	20.4	27.0	5.875	14.525	21.125	271.3	1.6
102	44.8	19.1	28.6	6.025	13.075	22.575	271.3	1.6
106	46.1	18.3	30.8	6.725	11.575	24.075	271.3	1.8
110	47.4	17.4	31.5	6.625	10.775	24.875	270.3	1.8
114	48.6	16.8	33.0	7.075	9.725	25.925	270.3	1.9
118	49.7	15.5	32.4	6.125	9.375	26.275	270.3	1.7
122	50.9	15.4	33.25	6.5	8.9	26.75	270.3	1.8
126	52.3	13.6	35.5	6.725	6.875	28.775	270.3	1.8
128	53.1	6.4	35.0	2.875	3.525	32.125		
130	53.9	0.0	36.1	0.225	-0.225	35.875		

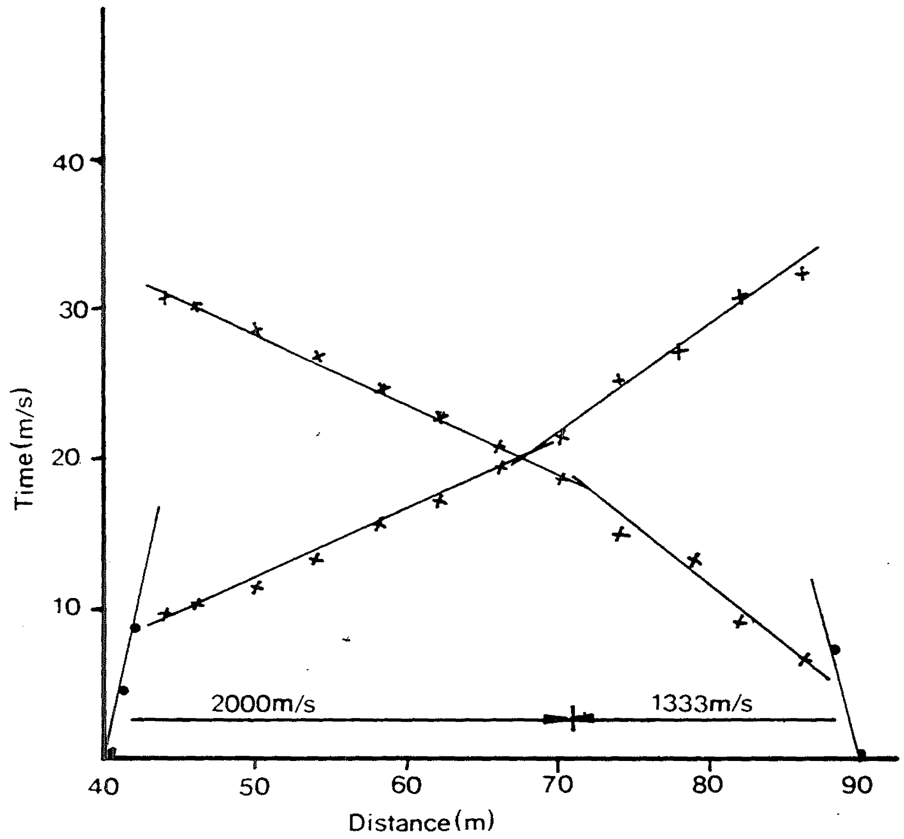
Results and calculations of seismic refraction traverse 120 - 150m.

Station	R.L.	T_f	T_r	T_d	$T_f - T_d$	$T_r - T_d$	F	d
120	50.3	33.8	0.0	0.1	33.7	-0.1		
121	50.6	30.7	2.1	-0.4	31.1	2.5		
122	51.0	28.6	5.6	0.3	28.3	5.3		
124	51.6	33.8	7.6	3.9	29.9	3.7	253	0.987
126	52.3	33.8	9.8	5.0	28.8	4.8	253	1.3
130	53.9	36.4	16.4	9.6	26.8	6.8	253	2.4
134	55.6	32.8	18.6	8.9	23.9	9.7	253	2.3
138	57.2	25.2	17.7	4.65	20.55	13.05	253	1.2
142	59.6	26.0	28.8	10.6	15.4	18.2	274	2.9
146	61.7	14.0	31.0	5.7	8.3	25.3	274	1.6
148	62.7	6.0	33.0	2.7	3.3	30.3		
150	63.1	0.0	33.4	-0.1	0.1	33.5		

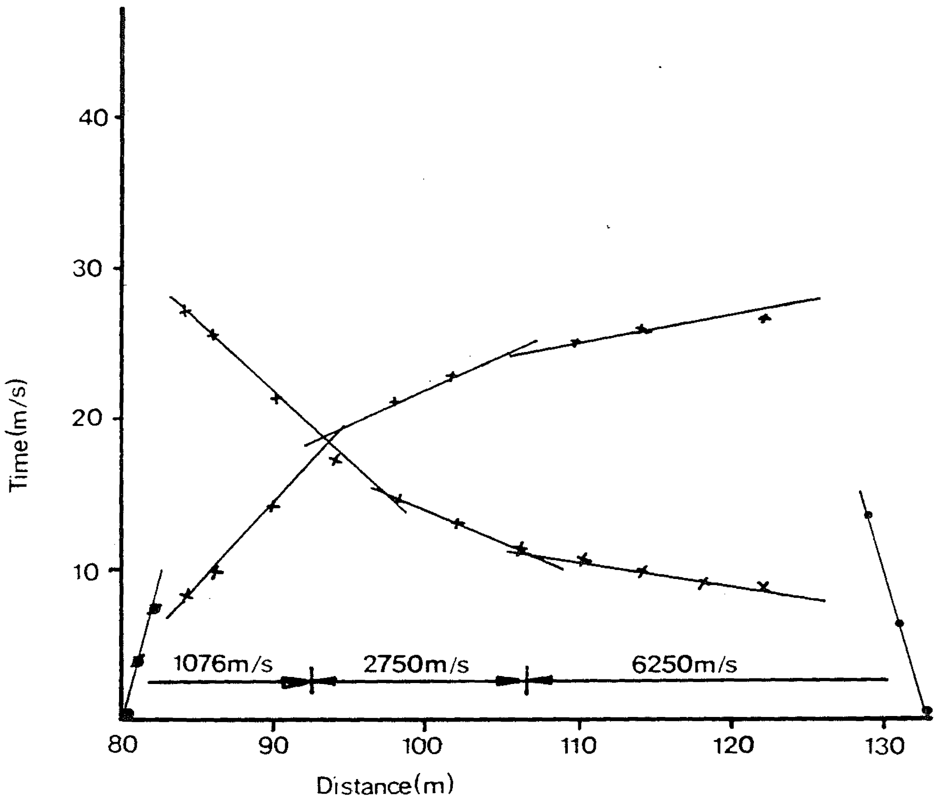
Time Distance Graph 0-50m



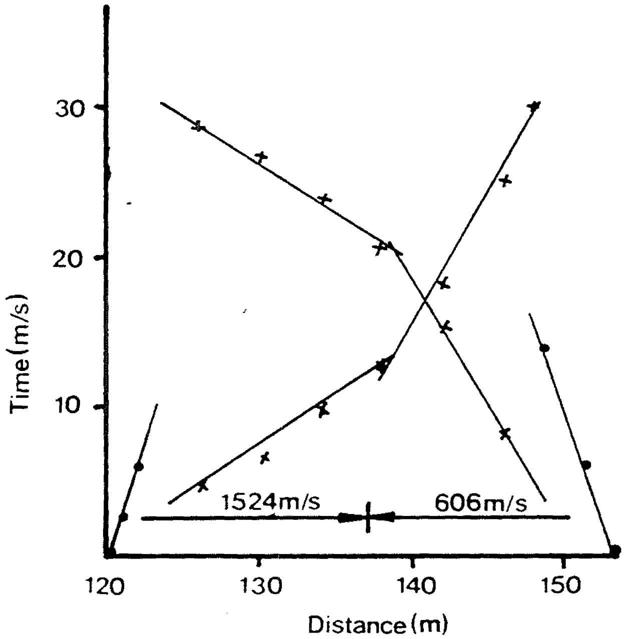
Time Distance Graph 40-90m



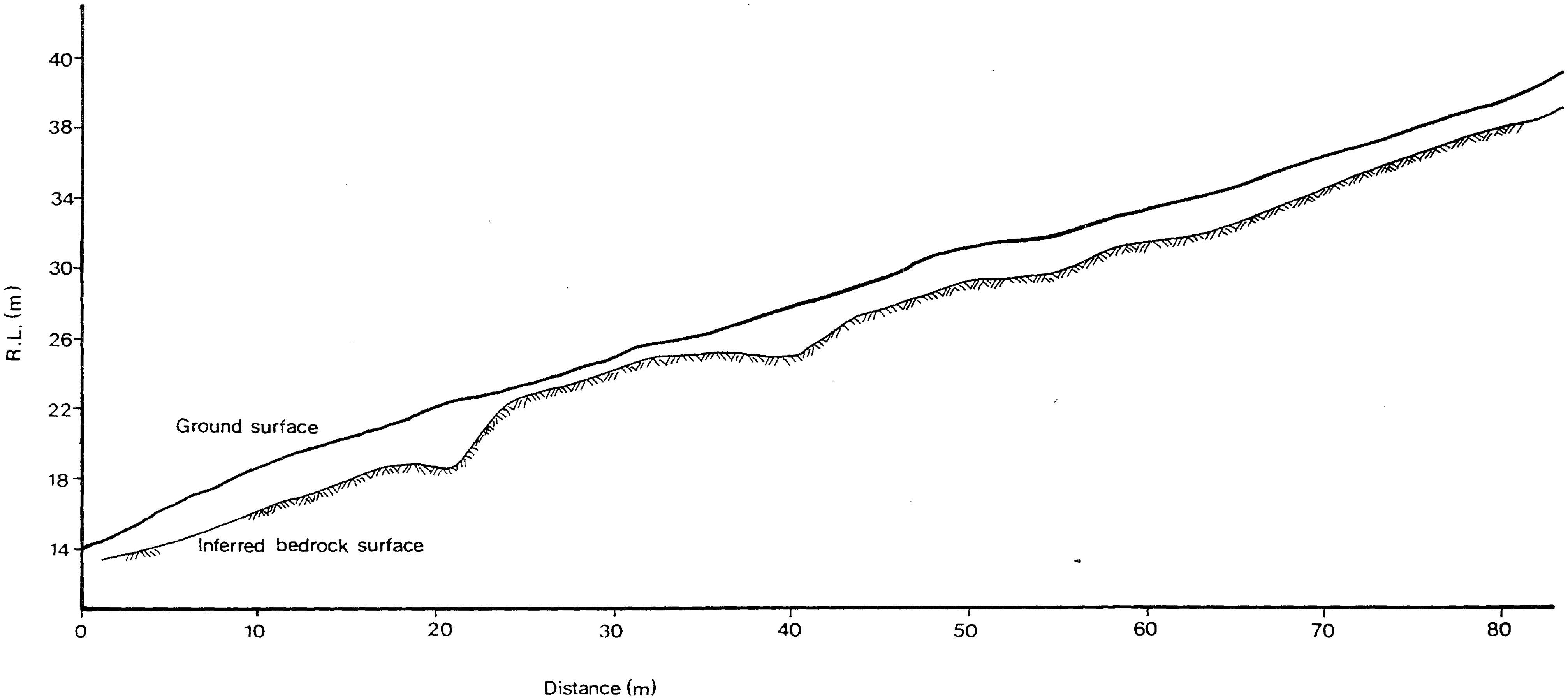
Time Distance Graph 80 - 130m



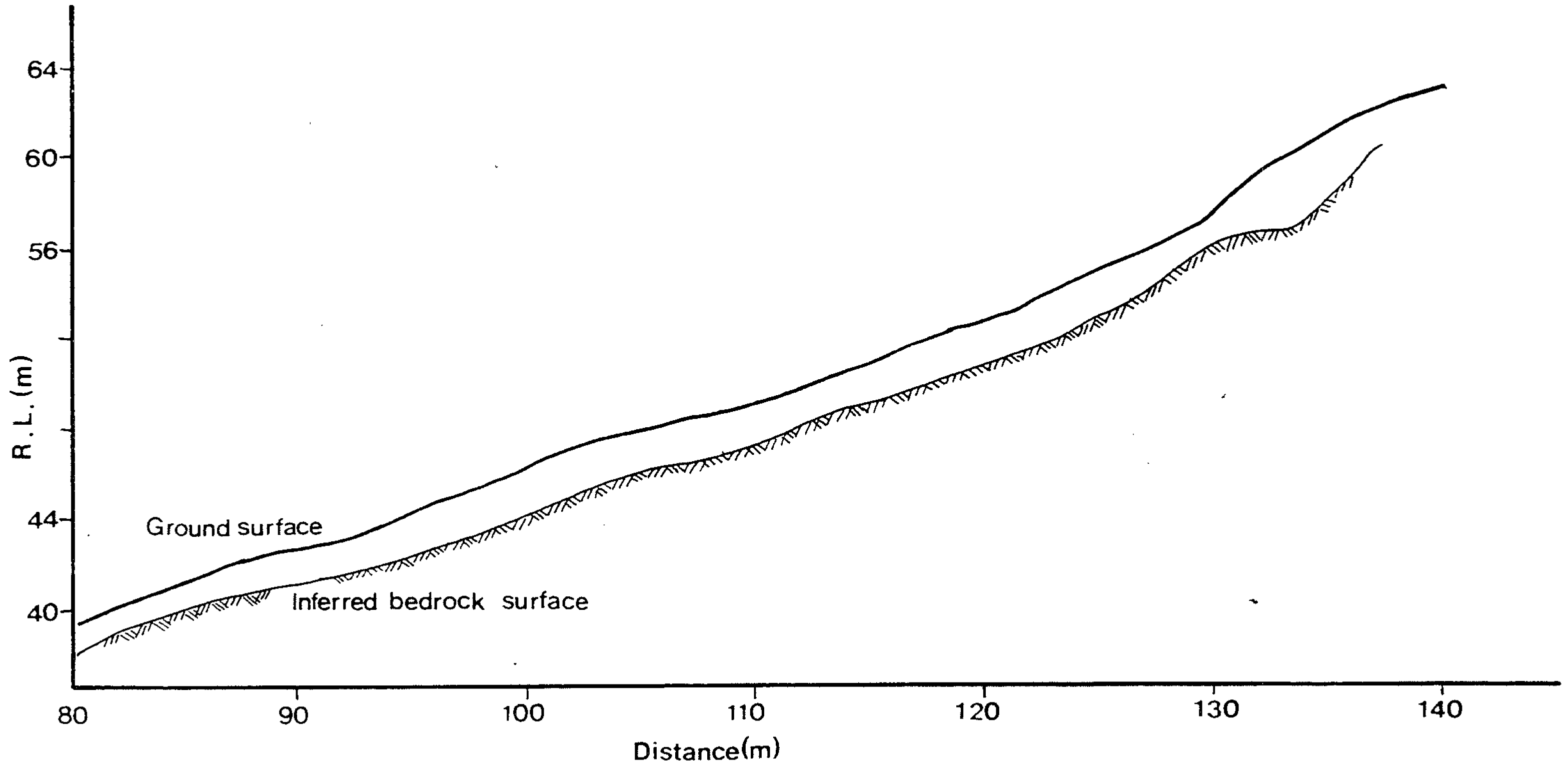
Time Distance Graph 120-150m



Plotted Bedrock Profile 0-83m



Plotted Bedrock Profile 80-140m



A3.4 SURVEY NETWORK.

A3.4.1 METHODOGOGY.

A Wild T1000 Theodolite combined with a Wild DI100 Distomat (EDM) was used to determine the displacement of fixed survey pegs from the Karoro Cemetery (Figure 4.6). The Karoro Cemetery is elevated above the surrounding terrain and allowed observation of the lower half of the landslide over the buildings in the intervening distance.

A peg established in the Karoro Cemetery formed the base station for the EDM. To avoid disturbance of the peg by grass cutting operations, general vandalism and accidental disturbance by foot traffic, the peg was hammered to below ground level. A nail hammered flush with the top of the peg provided a centering point for successive surveys, thereby reducing errors during the setup. Centering was accomplished by using the optical plumb on the tribrach. A tripod set-up using a single prism also utilised an optical plumb centred over bench mark S44. This bench mark is at reduced level (R.L.) 8.3141 a.s.l. and forms part of the Nagahere Fundamental Network. The instrument was then centred on the prism and set to a zero bearing 0°00'00" and this constituted the base line for the survey.

Survey marks on the slide consisted of wooden stakes 0.4m in length. These were coated with fluorescent paint to allow rapid relocation by field staff. The location of the survey markers (Pt 1, Pt 2 ...) is shown in Figure 4.8.

Two survey lines were established across the slide, a lower line across the cleared portion of the slide and a line approximately 15m upslope. A reference point was established beside the O'Sullivan house. Conventional surface surveys advocate the importance of establishing reference points outside the slide mass. In practice this proved difficult for two reasons:

1) The hill side geometry in the northern margin prevented siting of points where they could be seen in an area that was uninvolved in landsliding.

2) The thick vegetation constrained the placement of monitoring points.

In many cases, it was necessary to undertake minor scrub clearing in order to obtain a clear line of sight between the EDM and the prism. Even with the removal of some of the trees, it was still necessary to use an extension pole with the staff in order to project the staff over the tree canopy. Rapid regeneration of the vegetation meant that minor pruning had to be undertaken in each successive survey to maintain line of sight.

Repeated distance measurements were taken in order to obtain an "average" position for each survey marker, thereby minimising errors due to the movement of the staff pole. Surveys were repeated on approximately a monthly basis dependent on weather, equipment and staff availability, except the final survey which was spaced three months from the preceding survey. Five survey assistants were used in the field.

A3.4.2 ERRORS.

Goldwater (1990) has summarised both operator and inherent errors for this type of survey and for the EDM used. Atmospheric conditions (pressure and temperature) vary with time, therefore scale correction in ppm (parts per million) is required to compensate for changes during the length of the survey. Graphs for the atmospheric correction and correction for height above sea level are provided in the Wild DI1000 Operators Manual. An inbuilt error of $\pm$ 5ppm is present within the instrument.

Operator errors form the main source of error in surveying. These occur during the setup of the EDM and from movement of the staff pole during distance measurements.

The combination of atmospheric, height above sea level, inherent and operator errors, forms the total error for the survey

A3.4.3 DATA MANIPULATION.

The raw data obtained from the EDM is presented in polar coordinate form. For the purpose of graphically illustrating the data, it is useful to convert the raw data to rectangular coordinates. The data obtained during surveying was reduced to height a.s.l. (R.L.) and converted to rectangular coordinates using the following method. The R.L. for the survey peg was obtained by the following formula:

$$h \doteq x + h_i - h_s$$

Where h, x, h_i, and h_s are graphically illustrated in Figure A3.3.

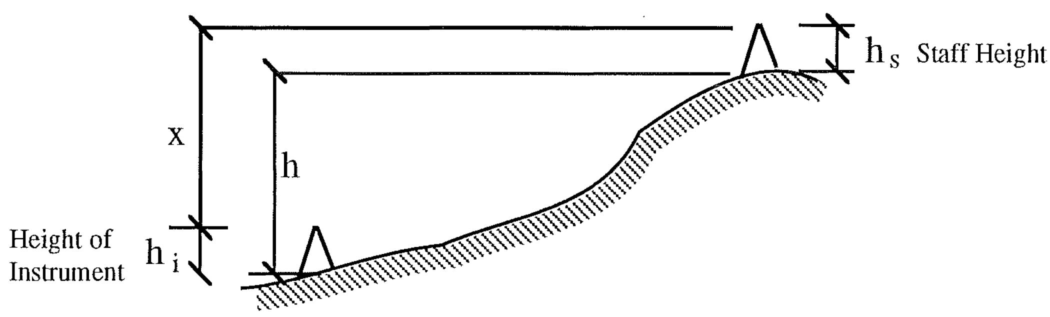


Figure A3.3 Diagramatic representation for the calculation of R.L.'s.

Horizontal bearing and distance to the peg (polar coordinates) were converted to rectangular coordinates (x and y) using an inbuilt function on a FX 4000P Casio Scientific Calculator. The x and y coordinates for each point are based on a false origin, in this case, the EDM setup position, illustrated in Figure A3.4.

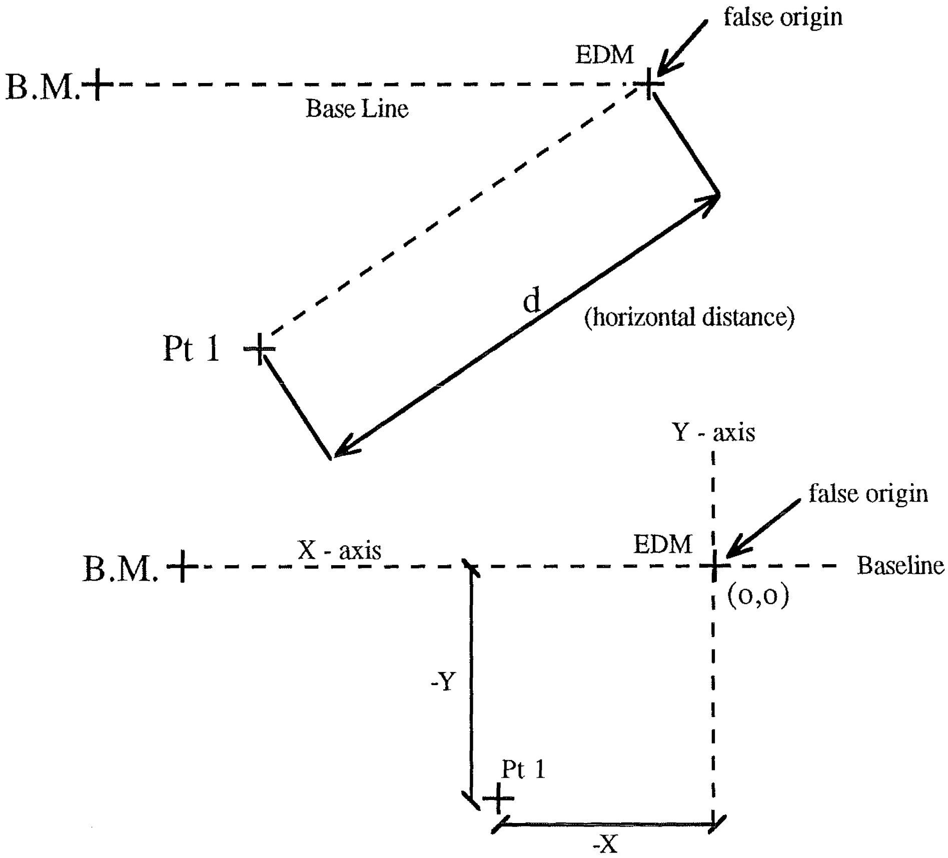


Figure A3.4 Methodology for the determination of rectangular coordinates. Redrawn from Goldwater (1990).

A3.4.4 RESULTS.

The manipulated data for each survey point is presented in Table A3.1 and graphically illustrated in Figure 4.11. Data for Pt 4 has not been presented as the peg was repeatedly removed by persons unknown between surveys. In addition, errors for each point have not been calculated as the recorded movement is considered insignificant and random in orientation.

Table A3.1 Survey Data Summary

Pt 1

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	34.026	271.224	60°08'00"	-135.065	235.202
13/10/92	34.035	271.229	600844	-135.017	235.235

14/11/92	34.033	271.227	600841	-135.02	235.231
12/12/92	34.04	271.225	600855	-135.003	235.239
16/1/93	34.04	271.238	600855	-135.009	235.25
18/02/93	34.051	271.229	600855	-135.005	235.242
23/05/93	34.057	271.227	600849	-135.012	235.237

Pt 2

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	34.082	259.186	70°47'02"	-85.307	244.745
13/10/92	34.079	259.219	704720	-85.296	244.784
14/11/92	34.086	259.216	704712	-85.305	244.778
12/12/92	34.088	259.218	704721	-85.294	244.783
16/1/93	34.097	259.214	704720	-85.294	244.779
18/02/93	34.086	259.217	704717	-85.299	244.781
23/05/93	34.087	259.217	704720	-85.295	244.782

Pt 3

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	34.154	253.823	80°16'50"	-42.851	250.18
13/10/92	34.169	253.917	801731	-42.818	250.281
14/11/92	34.159	253.908	801721	-42.828	250.27
12/12/92	34.172	253.917	801733	-42.815	250.281
16/1/93	34.167	253.911	801722	-42.827	250.273
18/02/93	34.164	253.921	801733	-42.816	250.285
23/05/93	34.164	253.913	801733	-42.814	250.277

Pt 5

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	23.563	228.588	64°09'14"	-99.654	205.722
13/10/92	23.564	228.6	640900	-99.673	205.726
14/11/92	23.533	228.587	640903	-99.665	205.716
12/12/92	23.575	228.601	640619	-99.655	205.736
16/1/93	23.577	228.597	640611	-99.661	205.729
18/02/93	23.571	228.606	640910	-99.666	205.736
23/05/93	23.567	228.59	640914	-99.655	205.724

Pt 6

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	23.572	221.627	73°36"34"	-73.581	209.056
13/10/92	23.571	221.614	703640	-73.571	209.047
14/11/92	23.576	221.606	703634	-73.574	209.036
12/12/92	23.579	221.614	703652	-73.559	209.05
16/1/93	23.576	221.615	703647	-73.564	209.05
18/02/93	23.573	221.618	703643	-73.569	209.051
23/05/93	23.571	221.612	703644	-73.566	209.045

Pt 7

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	23.409	225.91	80°29'13"	-37.337	222.803
13/10/92	23.418	225.907	803116	-37.203	222.823
14/11/92	23.221	225.903	803116	-37.203	222.819
12/12/92			No Reading Taken		
16/1/93	23.427	225.912	803116	-37.204	222.828
18/02/93	23.426	225.927	803108	-37.215	222.841
23/05/93	23.423	225.901	803109	-37.21	222.815

Pt 8

Date	R.L.	Distance	Angle	X Coord	Y Coord
21/9/92	74.508	358.036	73°39'34"	-100.732	343.574
13/10/92	74.526	358.054	733951	-100.709	343.599
14/11/92			No Reading Taken		
12/12/92			No Reading Taken		
16/1/93	74.529	358.053	733942	-100.724	343.594
18/02/93	74.523	358.052	733941	-100.725	343.592
23/05/93	74.514	358.052	733955	-100.702	343.599

A3.5 TENSION CRACK MONITORING.

A3.5.1 METHODOLOGY.

In the head scarp area crack monitoring consisted of sets of two wooden stakes, one either side of the crack (Figure A3.5). Nails were hammered into the top of each stake and the distance between the two nails was determined using a steal tape measure. If the tension crack was opening, then a net increase in the distance between the two pegs would be recorded in successive surveys. If the tension crack was found to be closing then a net decrease in the distance between the pegs would be recorded.

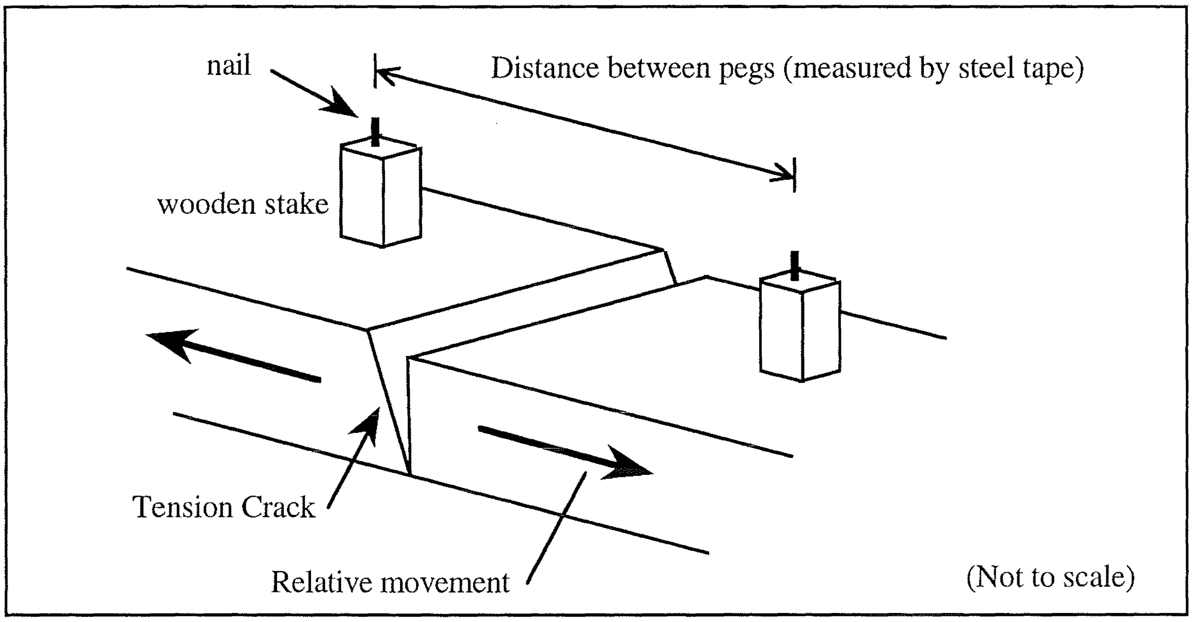
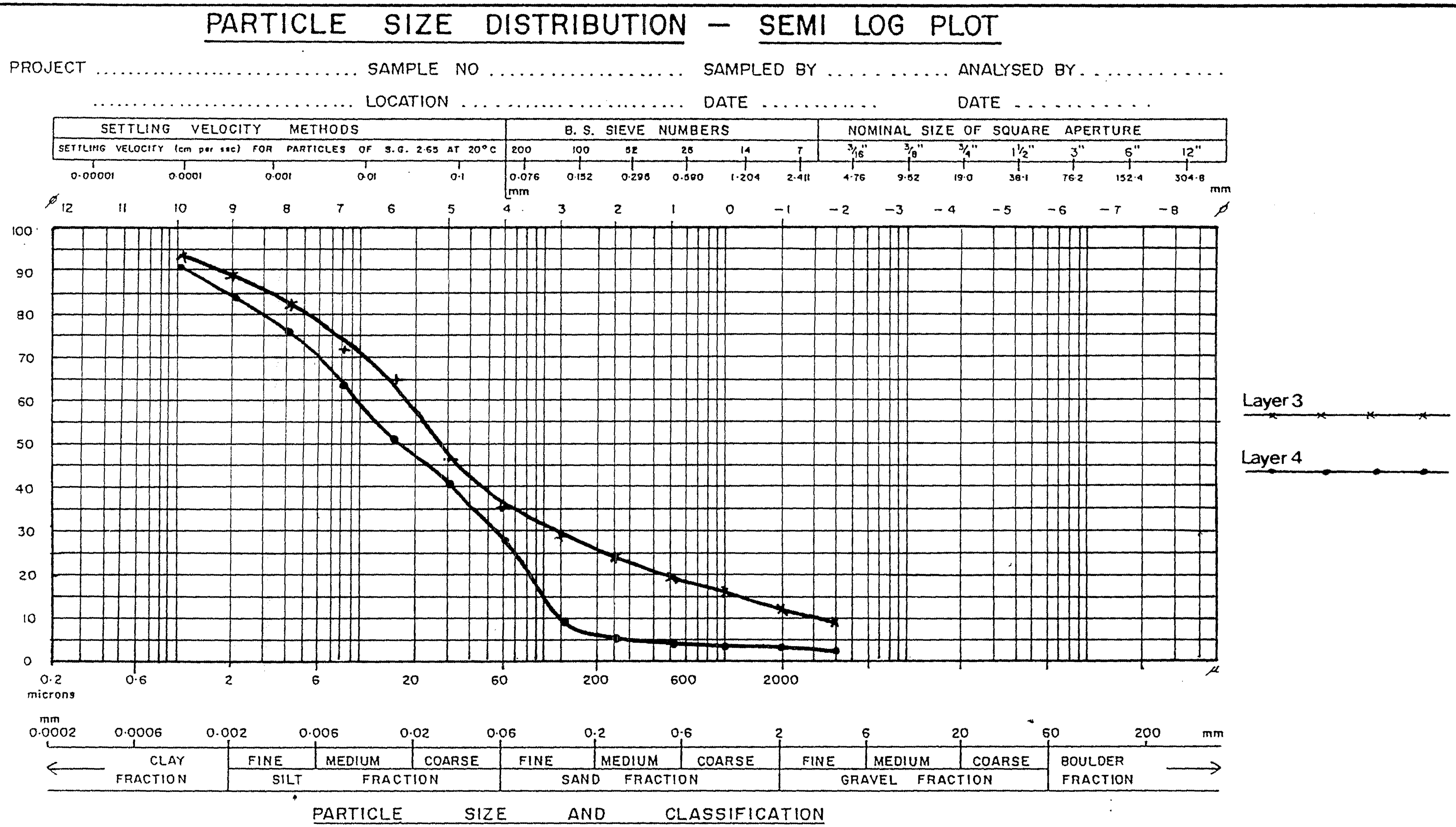


Figure A3.5. Diagramatic representation of the tension crack monitoring setup in the head scarp area of the Australasian Hotel Slide

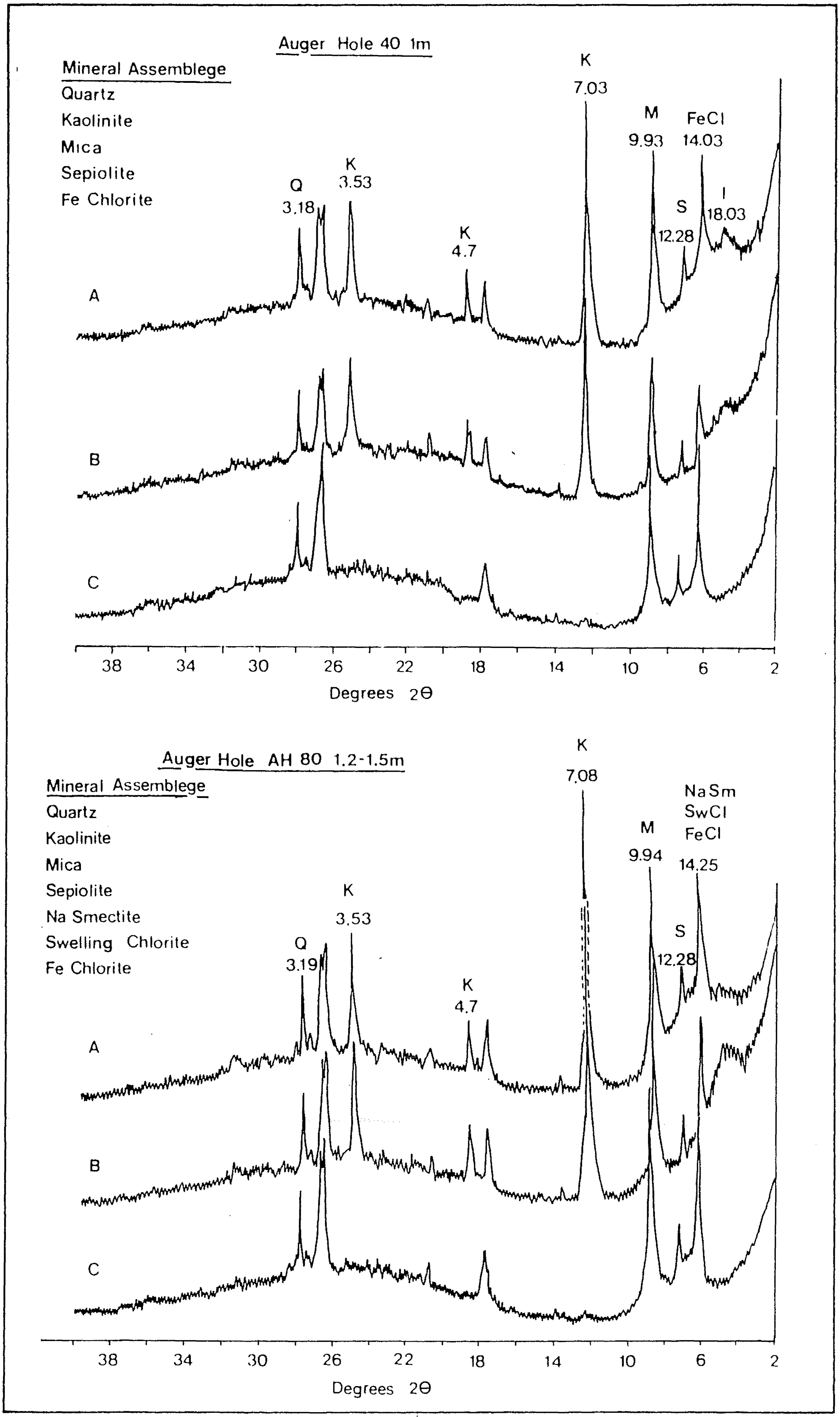
Monitoring points in the northern margin of the slide were based on a similar set-up to the head scarp area. The difference was in the purpose of the monitoring, in this case to record relative movement of suspected adjacent blocks of slide debris. The stakes were orientated obliquely across the lateral shear. An increase in displacement recorded across the pegs indicates that the block containing the downslope peg was moving faster than the adjacent block. If the net displacement between the pegs was decreasing then the slide block containing the upslope peg was moving faster than the adjacent block. If no net displacement was recorded then either no movement was occurring, or the two slide blocks were moving at the same speed. The position of the tension crack monitoring points is shown in Figure 4.8.

A3.5.2 RESULTS.

The results are illustrated and discussed in Section 4.5.5.



AUSTRALASIAN HOTEL SLIDE SAMPLES



Auger Hole 40 1m
Mineral Assemblege
Quartz
Kaolinite
Mica
Sepiolite
Fe Chlorite
K 7.03
M 9.93
FeCl 14.03
Q 3.18
K 3.53
S 12.28
I 18.03
K 4.7
A
B
C
38 34 30 26 22 18 14 10 6 2
Degrees 2Θ
Auger Hole AH 80 1.2-1.5m
Mineral Assemblege
Quartz
Kaolinite
Mica
Sepiolite
Na Smectite
Swelling Chlorite
Fe Chlorite
K 7.08
NaSm
SwCl
FeCl
14.25
M 9.94
K 3.53
Q 3.19
S 12.28
K 4.7
A
B
C
38 34 30 26 22 18 14 10 6 2
Degrees 2Θ

APPENDIX 4.

A4.1 TERMINOLOGY.

The Resource Management Act (1991) defines a "Natural Hazard" as:

"*...any atmospheric or earth or water related occurrence (including earthquake, tsunami, erosion, volcanic, and geothermal activity, landslip, subsidence, sedimentation, wind, drought, fire or flooding) the action of which adversely affect human life, property, or other aspects of the environment*"

Table A4.1 Relevant definitions of hazard and risk from the literature

VARNES (1984)	IPENZ (1983)	BRITISH STANDARD 4778	EINSTEIN (1988)
HAZARD: The probability of occurrence of potentially damaging phenomenon within a specified time period and area (H) VULNERABILITY: Degree of loss to a given element (or set) resulting from occurrence of given magnitude event (V). SPECIFIC RISK: Expected degree of loss due to a particular natural phenomenon. (R_s) ELEMENTS AT RISK: The population, properties, economic activities etc. at risk in a given area (E). TOTAL RISK: Expected loss of life, injuries, property damage due to specified natural phenomenon (R_t). Where R_t=E R_s=E*(H*V)	HAZARD: A condition or situation which has the potential to create harm or increase harm to people property or the environment. RISK: The probability a potential hazard will be realised and the probability of harm itself.	HAZARD: A set of conditions in the operation of a product or system with the potential for an initiating an accident sequence. RISK: The combined effect of the probability of occurrence of an undesirable event and the magnitude of the event.	DANGER: Existing or potential natural phenomenon affecting a specified area. HAZARD: Probability that a specified danger occurs within a given period of time. RISK: The product of hazard times the potential worth of loss where loss includes death, injury, or capital loss or non-monetry environmental effects.

A4.2 A REVIEW OF HAZARD ZONATION SCHEMES CURRENTLY IN USE.

A4.2.1 INTRODUCTION.

This section is a brief review of some of the most commonly used hazard zonation systems in use. The purpose of this section is to outline some of the other approaches that are used both here and overseas.

A4.2.2 RELATIVE SLOPE STABILITY ASSESSMENT, CALIFORNIA.

The landslide susceptibility maps developed by Brabb *et al.* (1972) for the San Mateo County, California, provide a measure of the "relative" susceptibility of slope materials to landsliding. The approach taken is as follows: the area of outcrop for each geologic unit in the county was estimated using a grid system. A landslide inventory map was then superimposed on the geologic map, and the percentage area of each unit affected by landsliding was calculated. From this data, each geologic unit could be assigned a landslide susceptibility class. By overlaying a slope map, the map unit containing the maximum frequency of landslides could be evaluated. Those slope intervals with the maximum landslide frequency were assigned the same susceptibility rating as the underlying bedrock unit. The susceptability rating was progressively decreased for areas falling within slope intervals showing fewer landslides.

In much the same approach, Nilsen *et al.* (1979) used existing geologic and slope maps, and the interpretation of aerial photographs, in an assessment of the San Fransico Bay region at a scale of 1:125 000. Generalised maps of slope range were combined with maps of landslide deposits to create four preliminary map units of relative slope stability. These were then combined with a map of susceptible bedrock and surficial geologic units to define six units of the final relative stability map. The classification criteria are summarised in Table A4.2.

Table A4.2 Principle criteria for slope-stability classification used by Nilsen *et al.* (1979) in 1:125000 maps of the San Fransico Bay region

<table>
<tr><th></th><th colspan="3">Slope</th></tr>
<tr><th></th><th><5%</th><th>5 - 15%</th><th>>15%</th></tr>
<tr><td>No landslide deposits</td><td rowspan="2">1
Stable</td><td rowspan="2">2
Generally stable</td><td>3
Moderately stable</td></tr>
<tr><td>Susceptible bedrock</td><td>4
Moderately unstable</td></tr>
<tr><td>Susceptible surfical deposits</td><td>1A
Subject to liquefaction</td><td colspan="2">None</td></tr>
<tr><td>Landslide deposits</td><td colspan="3">5
Unstable</td></tr>
</table>

A4.2.3 GEOTECHNICAL AREA STUDIES PROGRAMME - HONG KONG.

The Geotechnical Control Office (GCO) developed the Geotechnical Area Studies Programme (GASP) to cope with the physical constraints to urban development in Hong Kong (Brabb 1988). Landslide investigation and mitigation is a function of the GASP, under which a terrain evaluation approach has been adopted. It consists of:

a) Regional studies at a scale of 1:20 000 (Table A4.3) based on aerial photographic interpretation, site reconnaissance and existing data;

b) District studies (stage 1) - concentrated geotechnical assessments at a scale of 1:2500 based on API, site reconnaissance and existing data (Table A4.4); and

c) District Studies (Stage 2) - In-depth geotechnical assessments based on the results of Stage 1 studies (Brand 1988).

A Terrain Classification Map forms the basis of the GASP investigation programme. It is a storage map from which a large number of derivative maps (Figure A4.1) can be obtained for a range of planning purposes. An interpretative Geotechnical Land Use Map (GLUM) is developed from the Terrain Classification Map. This divides the terrain into four classes on the basis of increasing geotechnical limitations to development (Table A4.5). The production of a General Limitations and Engineering Appraisal Map (GLEAM) forms the end result in the terrain evaluation of the Hong Kong area. The GLEAM map is developed from regional studies and is designed specifically as a planning tool to aid in the identification of land areas suitable for future development (Brand 1988).

(a) Slope Gradient	Code
0 -5°	1
5 - 15°	2
15 - 30°	3
30 - 40°	4
40 - 60°	5
>60°	6

(c) Erosion and Instability		Code
Sheet erosion	- minor	1
	- moderate	2
	-severe	3
Rill erosion	- minor	4
	- moderate	5
	-severe	6
Gully erosion	- minor	7
	- moderate	8
	-severe	9
Well-defined recent landslip, >1 ha in size		a
General instability	-relict	n
	-recent	r
Coastal instability		w

(b) Terrain Component		Code
Hillcrest or ridge		A
Side slope	-straight	B
	-concave	C
	-convex	D
Footslope (colluvium)	-straight	E
	-concave	F
	-convex	G
Drainage plain (colluvium)		H
Floodplain		I
Coastal plain		J
Litoral zone		K
Rock outcrop		L
Cut	-straight	M
	-convave	N
	-convex	O
		P
Fill	-straight	R
	-concave	S
	-convex	T
General disturbed terrain		V
Alluvial plain		X
Reclamation		Z
Waterbodies:		
Natural stream		1
Man-made channel		2
Water storage		3
Fish pond		4

Table A4.3 Terrain classification schedule for the regional GASP studies (1:20 000 scale). from Brabb (1988).

Table A4.5 Definition of the GLUM classes used by the GASP programme. From Brand (1988).

GLUM Class Characteristics	Class 1	Class 2	Class 3	Class 4
Geotechnical Limitations	Low	Moderate	High	Extreme
Suitability for Development	High	Moderate	Low	Probably unsuitable
Engineering Costs for Development	Low	Normal	High	Very high
Intensity of Site Investigation Required	Normal	Normal	Intensive	Very intensive
Examples of Terrain in GLUM Class	1. Insitu terrain <15°, minor erosion. 2. Cut platforms in insitu terrain. 3. Cut slope <15°, <30m high in insitu terrain.	1. Insitu terrain 15-30°, no instability or severe erosion. 2. Insitu terrain <15°, severe erosion. 3. Colluvium <15°, no instability or severe erosion.	1. Insitu terrain 30-60°, no instability or severe erosion. 2. Insitu terrain<15°, history of landslips. 3. Colluvium <15°, general instability.	1. Insitu terrain >60°. 2. Insitu terrain 30-60°, instability or severe erosion. 3. Colluvium 30-60°, moderate erosion.

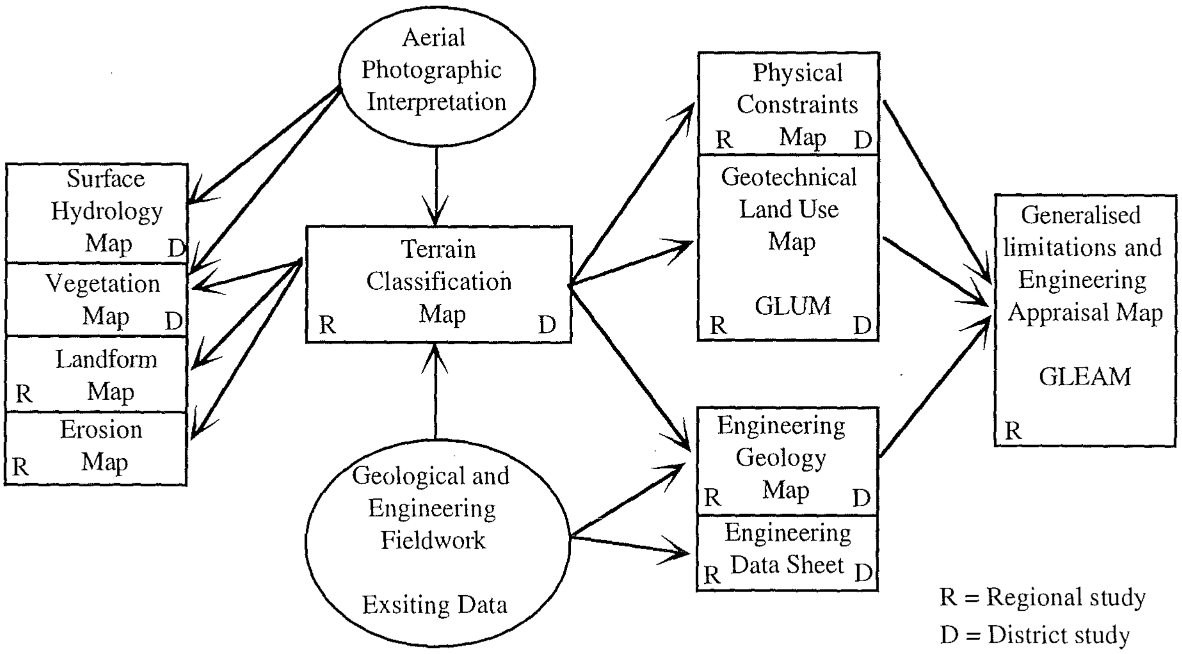


Figure A4.1 Derivation of Maps during GASP studies. From Brand (1988).

(a) Slope Gradient	Code
0 - 5°	A
5 - 15°	B
15 - 30°	C
30 - 40°	D
40 - 60°	E
>60°	F

(b) Terrain Component	Code
Hillcrest	1
Ridge	2
Sideslope	3
Footslope	4
Drainage plain	5
Incised drainage channel	6
Disturbed terrain	7
Disturbed terrain/drainage	8
Rock exposure	9
Alluvial plain	10
Floodplain	11
Coastal plain	12
Stream course (perennial)	13
Stream course with rock exposure in bed (perennial)	14
Stream course with rock exposure in bed (ephemeral)	15
Man-made channel (including catchwaters)	16
Water storage	17
Swamp/marsh	18
Reclamation	19
Beach	20
Dune	21

(c) Terrain Morphology	Code
Straight - insitu terrain	a
Concave - insitu terrain	b
Convex - insitu terrain	c
Straight - colluvium	d
Concave - colluvium	e
Convex - colluvium	f
Straight - <2m thick colluvium	g
Concave - <2m thick colluvium	h
Convex - <2m thick colluvium	i
Straight - corestones on insitu	j
Concave - corestones on insitu	k
Convex - corestones on insitu	l
Straight - alluvium	m
Concave - alluvium	n
Convex - alluvium	o

(d) Erosion and Instability	Code
No appreciable erosion	1
Well defined landslip - intergral	2a
- scar	2b
- debris	2c
General instability	2d
- colluvium	2e
Sheet erosion - insitu	3a
	3b
- minor (1-10% bare)	3c
Rill erosion - moderate (10-40%)	4a
- severe (>40%)	4b
	4c
Gully erosion - minor	5a
- moderate	5b
- severe	5c
Highly jointed rock exposure, - minor	6
or boulders - moderate	
- severe	

(e) Slope Condition	Code
Cut, 0 - 5m	1
5 - 10m	3
10 - 20m	5
20 - 30m	7
>30m	9
Fill,	2
0 - 5m	4
5 - 15m	6
15 - 30m	

(f) Hydrology	Code
Inlet zone	a
Outline zone	b
Water ponding area	c
Subsurface flow	d

(g) Vegetation	Code
Grassland	1
Shrubland	2
mixed broadleaf woodland	3
Cultivated land	4
Non-vegetated	5
- soil	6
- rock	7
reafforestation - built upon	8

Table A4.4 Terain classification schedule for the District GASP Studies (1:2500 scale). From Brand (1988).

A4.2.4 ZERMOS: FRANCE.

The ZERMOS (Zones Exposed to Risk of Soil Movement) programme is one of the most well developed approaches to risk mapping in use. The following case study based on Meneroud and Calvino (1976) outlines the methodology. The mapped area is divided into four zones, shown by colours according to the level of risk. Types of movement and direction, activity, and sites of erosion are shown by graphic symbols. In evaluating risk for each type of movement the following "determining factors" were considered: lithology, structure, slope morphology and hydrogeology. The level of risk is then fixed according to the presence of unfavourable values of conditions of the determining factors. Up to seven hazard classes are used in the Zermos maps (scale 1:25 000) which are accompanied by a detailed text summary. It should be noted that the French term "risque" (risk) is equivalent to the term "hazard" used in the English language.

The ZERMOS system is often extended to the production of 1:5 000 Plans of Risk Exposure (PER) which are legal, technical and cartographic documents of reference with regard to land use and building rules (Flageollet 1989).

A4.2.5 THE PUCE SYSTEM.

The PUCE (Pattern-Unit-Component-Evaluation) system was developed in Australia for the classification of terrain. The process of terrain analysis involves the classification of land areas on the basis of similarity or homogeneity of certain attributes, and assesses (qualitatively) or evaluates (quantitatively) like areas for the properties of the terrain that are significant for the desired purpose (Finlayson 1984). The PUCE system classifies terrain on the basis of characteristics such as the engineering properties of the underlying rock and soil, the slope of the land and its vegetation (Grant and Finlayson 1978).

The PUCE system is hierarchical in that members of each class are composed of an association of members from the preceding class. The highest and most general of the components is the "province", which is defined by geology and age. A terrain "pattern" forms an integral part of a province and has consistent local relief amplitude, a characteristic drainage pattern and uniform drainage density. A terrain "unit" is defined as an area which has a single landform with a characteristic soil association and vegetation formation. A terrain "component" is the most detailed and is defined on the basis of slope angle and form, soil horizons, land use/cover and vegetation (Finlayson 1984).

A hazard assessment for various processes based on the physical attributes or constraints an any area is then made. This in turn allows for the adminstration of land-use planning.

A4.2.6 URBAN LAND USE CAPABILITY SURVEYS, NEW ZEALAND.

Urban land use capability (ULUC) surveys have been used in New Zealand to provide local authorities with an integrated assessment of the physical factors relevant to urban planning (Jessen 1987). The need for ULUC surveys arose from the responsibilities and liabilities imposed on local authorities by the Town and Country Planning Act 1977 and the Local Government Amendment Acts 1978 and 1979 (Lawerence 1981). ULUC surveys are prepared according to strict guidelines (Jessen 1987) and comprise four types of information:

1) Descriptive physical information (rock, soil, landform, erosion, drainage, land cover/land-use).

2) A division of the land according to similar physical features, i.e., a set of boundaries which define map units.

3) Interpretations of the physical information within each map unit, identifying the main types of constraint and the degree to which each could limit the development and use of the land.

4) A ULUC assessment, which brings together information in 1 and 3 for the area of land defined in 2. It is a general assessment.

The ULUC assessment (Figure A4.2) involves the compilation of an "inventory of physical factors" (1 above). These physical factors are recorded for each "inventory map unit" and are considered to be homogeneous within that map unit at that scale of mapping (2 above). An assessment of the physical constraints to urban land development and use is made. This is recorded in a numerically ranked "constraint code". The constraint code comprises "predictive" information which is in contrast to the "factual" inventory information (Jessen 1987).

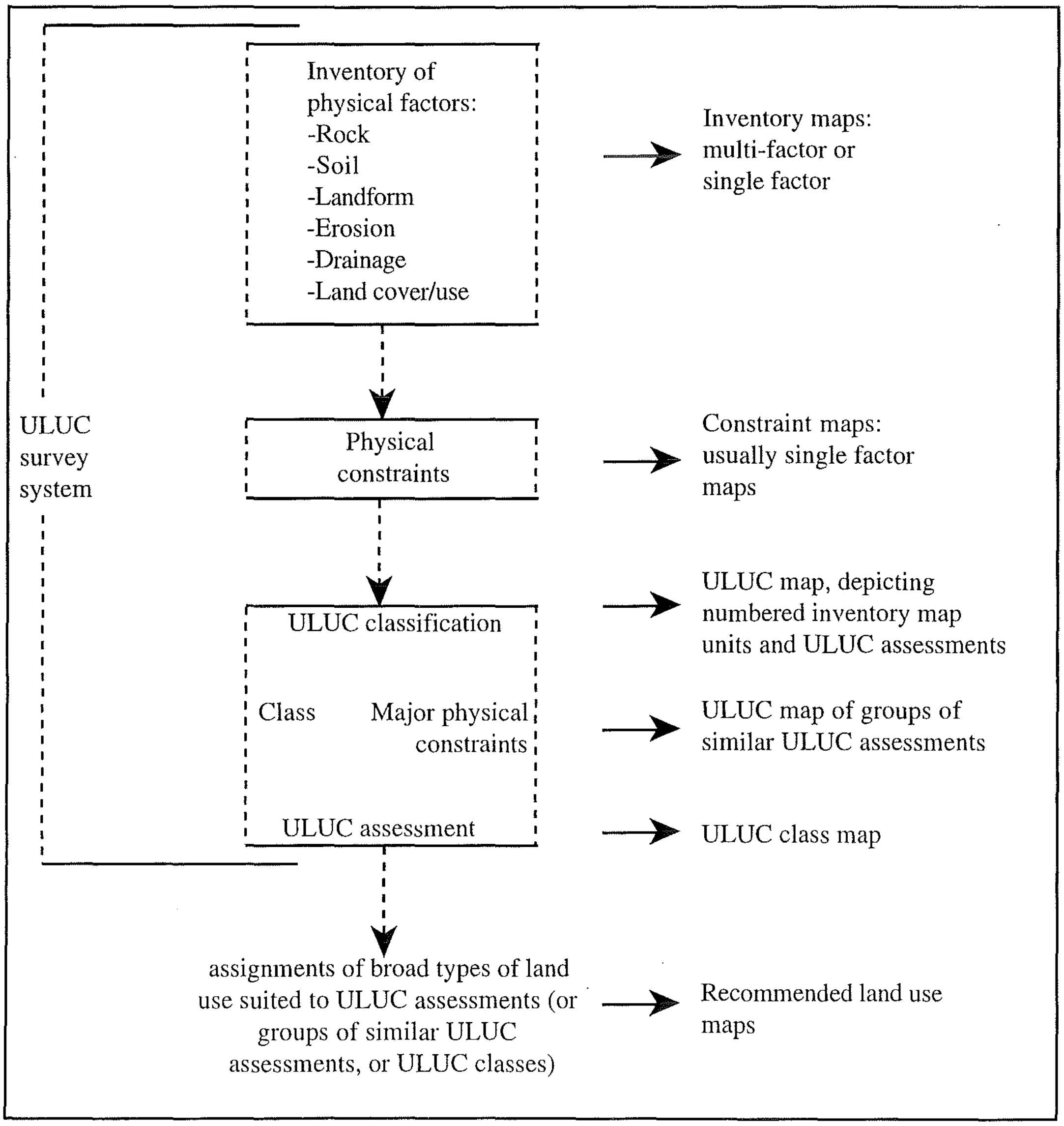


Figure A4.2 Outline of the ULUC survey system and process of ULUC assessment. From Jessen (1987).

The assessment of "urban land use capability" (4 above) is the final step in the ULUC survey system. The ULUC assessment is defined as:

"*the overall degree and dominant type of physical constraint which together determine the land's capability for urban development and sustained urban use*" (Jessen 1987).

The assessment comprises a ULUC class (Figure A4.3) which defines the overall degree of constraint.

Table A4.6 Definition of the ULUC classes. From Jessen (1987).

<u>ULUC class A</u> -	Land with negligible, or no, physical constraints to urban development and use.
<u>ULUC class B</u> -	Land with slight physical constraints to urban development and use.
<u>ULUC class C</u> -	Land with moderate physical constraints to urban development and use.
<u>ULUC class D</u> -	Land with severe physical constraints to urban development and use.
<u>ULUC class E</u> -	Land with physical constraints so severe that they essentially preclude any kind of urban development.

The scale (Table A4.7) at which the ULUC survey is carried out is dependent upon the planning and development requirements of the requesting local authority.

Table A4.7 Scales and applications of ULUC surveys

Scale term →	RECONNAISSANCE	SEMI-DETAILED	DETAILED
Scale →	1:25,000-1:15,000	1:15,000-1:8,000	1:8,000-1:1,000
APPLICATIONS			
	A basis for the preparation and review of regional and district planning schemes	A basis for the preparation and review of district planning schemes	A basis for the preparation and appraisal of concept and scheme plans of subdivisions
	Identification of potential urban areas and of physical constraints within them	Preferred location of various urban land uses	Actual location of various urban land uses

ULUC surveys have been widely used in New Zealand including the Greymouth area. This was compiled by Hutchinson and Mckie (1979) at a scale of 1:4000. The survey was based on soil conservation principles and included an assessment of the climate, geology and physiography, soils, landform (slope and terrain), erosion rating, existing land use and vegetation. The survey area which included the coastal plain (north and south of Greymouth) and the Sawyers Creek alluvial flat, divided the land into seven classes based on an assessment of the physical factors listed above. Each class represents landscape stability, flooding hazard and assessed potential for urban development. The survey has been incorporated into the District Scheme by the local authority.

A4.3 CHRONOLOGICAL HISTORY OF LANDSLIDES TO HAVE AFFECTED GREYMOUTH.

7 Feburary 1867

Heavy rain and an accompanying earthquake caused extensive slipping, damaging some very valuable gardens. A pleasent villa in Brook Street is scarcely safe to live in with an overhanging and unsupported mass of stones and soil threatening to overwhelm it (Grey River Argus (G.R.A.) 07/02/1867).

24 March 1871

A face of earth collapsed burying one man in the South Beach area. His mates quickly removed him but he died soon after (G.R.A. 24/03/71).

10 September 1873

A man named Francis Ryan was brought to Grey Hostipal suffering severe bruises caused when a slip of earth fell on him while he was working for M^c^Whiter and Co. in the gravel pit on the Marsden Road (G.R.A. 10/09/1873).

8 April 1874

Outside the town the heavy rains have seriously interfered with traffic on the roads, and heavy landslips have occurred up the Omoto road, in some places covering the railway lines (G.R.A. 08/04/1874).

9 April 1874

The road between Greymouth and Omotuomotu is crumbling down at the front and slipping at the back and soon will cease to be road at all. At one place a very extensive slip is slowly travelling down (G.R.A. 09/04/1874).

12-14 June 1921

Three days of steady rain caused slips to come down on roads and railways, but no extensive damage was reported.

18 June 1929

An earthquake occurring on the 16 June 1929 in the Murchison area caused 60,000-100,000 tonnes of rock to fall in the Cobden Quarry and some was thrown nearly 100m (G.E.S. 18/06/1929).

27 June 1934

Slipping occurred on some roads following 102mm of rain in 24 hours.

10-12 October 1936

Extensive slips blocked roads after a north westerly storm dropped 55mm of rain in a 30 hour period of which, 35mm was recorded in 6 hours.

8-9 January 1938

Greymouth recieved 52mm of rain in 24 hours and 77mm in 48 hours. Extensive slipping resulted and a train ran into a slip at Omoto near Greymouth causing considerable damage to the rolling stock.

21 March 1938

A total of 178mm of rain falling in four days combined with 44mm of rain falling in two hours during the succeeding day caused extensive landsliding and erosion on hillsides in the Greymouth area. The Greymouth - Omoto road was blocked by a 11500 cubic metre slip.

5 May 1942

A total of 160mm of rain was recorded at Greymouth in 48 hours. This caused extensive minor slipping in the low levels of Greymouth.

18 December 1942

Numerous landslides in Greymouth resulted from a total of 132mm of rain falling in 48 hours.

25-27 Febuary 1955

In twenty-four hours to 9.00 am on the 26th, 114mm of rain was recorded in Greymouth following 25mm the previous day. Landslides covered the railway line at Omoto.

14 Febuary 1958

Intense rain during the night (152mm in six hours) caused many slips in the Cobden and Greymouth areas. Mud and silt from numerous slips was deposited over a wide area in Greymouth and Cobden destroyed a number of sheds and damaged two houses (Greymouth Evening Star (G.E.S.) 15/02/1958).

13 March 1958

Heavy rain in the Greymouth area caused reactivation of the slips created during the Febuary event (G.E.S. 15/03/1958).

7-10 November 1961

Persistent rain around Greymouth caused slipping at the Cobden Quarry which closed State Highway 6 (G.E.S. 10/11/1961).

10 May 1962

An earthquake centred near Westport caused extensive slips around Omoto (G.E.S. 10/05/1962).

26-27 April 1966

164 mm of rain was recorded in Greymouth to 9.00 am on the 26th. This caused a severe slip in Freyberg Terrace which left a house overhanging. The slip material piled downslope onto a house in Rochford Street. A slip in Ashmore street in Cobden blocked a drain causing the water

to back up flooding a house. Landslides also occurred in Lydia Street and on Arnotts Heights (G.E.S. 27/04/1966).

9-10 April 1968

A total of 53mm of rain was recorded in Greymouth during the "Wahine Storm". Greymouth, Cobden and Runanga all received severe slipping (G.E.S. 09/05/1968).

6 May 1968

Greymouth Harbour received 31mm of rain in 24 hours while Karoro received 29mm of rain. Slips produced by the April storm were reactivated, one which blocked the Cobden Railway line (G.E.S. 06/05/1968).

27-28 December 1976

139mm of rain was recorded at Karoro in the 24 hour period to 9.00 am on the 28th. Minor slipping was recorded in Greymouth and at Omoto, a slip reduced traffic to one lane (G.E.S. 28/12/1976).

13-14 April 1978

State Highways and rail links north and south of Greymouth were impassable due to numerous slips. Stillwater Creek, blocked by a large slip, overflowed its banks and flooded a large part of lower Greymouth (Westland Catchment Board, File 377). A large slip occurred at Boddytown demolishing the residents back fence, glasshouse and garage and stopping only two feet from the house. A large slip on mudstone "greasyback" occurred at Hinton Road, Karoro (G.E.S. 14/04/1978).

28 November 1980

Small slips at Omoto resulted from a total of 64mm of rain falling in 24 hours (G.E.S. 28/11/1980).

2 December 1982

An electrical storm centred on Greymouth with 84mm of rain being recorded at the harbour in the 24 hour period to 9.00 am. Slipping occurred in Freyberg Terrace (G.E.S. 22/12/82) and a slip on the Cobden Hill blocked State Highway 6. State Highway 7 and the Midland Railway were blocked by slips at Omoto (G.E.S. 21/12/1982).

17-18 October 1984

Wide spread slipping occurred in the Greymouth County following 60mm of rain in the 12 hours to 9.00 am on the 18th (G.E.S. 18/10/1984)

13-14 September 1988

During this period a total of 169mm of rain was recorded at Greymouth. A large slip about 300m east of Cobden blocked State Highway 7 (G.E.S. 17/09/1988).

6-8 November 1989

45mm of rain at Greymouth resulted in a minor slip at the Greymouth approach to the Cobden Bridge (G.E.S. 17/09/1988).

15 December 1989

Rock falls occurred on the Greymouth side of the Cobden Bridge.

14-16 January 1990

Minor slips on the Greymouth side of the Cobden Bridge were caused by heavy rain (G.E.S. 15/01/1990).